VOLUME 85 NO. SA5
SEPTEMBER 1959
PART 1

JOURNAL of the

Sanitary Engineering Division

PROCEEDINGS OF THE

AMERICAN SOCIETY OF CIVIL ENGINEERS FOUNDED 1852

AMERICAN SOCIETY OF CIVIL ENGINEERS

BASIC REQUIREMENTS FOR MANUSCRIPTS

This Journal represents an effort by the Society to deliver information to the reader with the greatest possible speed. To this end the material herein has none of the usual editing required in more formal publications.

Original papers and discussions of current papers should be submitted to the Manager of Technical Publications, ASCE. Authors should indicate the technical division to which the paper should be referred. The final date on which a discussion should reach the Society is given as a footnote with each paper. Those who are planning to submit material will expedite the review and publication procedures by complying with the following basic requirements:

1. Titles should have a length not exceeding 50 characters and spaces.

2. A 50-word summary should accompany the paper.

3. The manuscript (a ribbon copy and two copies) should be double-spaced on one side of 8½-in. by 11-in. paper. Papers that were originally prepared for oral presentation must be rewritten into the third person before being submitted.

4. The author's full name, Society membership grade, and footnote reference stating present employment should appear on the first page of the paper.

5. Mathematics are reproduced directly from the copy that is submitted. Because of this, it is necessary that capital letters be drawn, in black ink, ⅛-in. high (with all other symbols and characters in the proportions dictated by standard drafting practice) and that no line of mathematics be longer than 6½-in. Ribbon copies of typed equations may be used but they will be proportionately smaller on the printed version.

6. Tables should be typed (ribbon copies) on one side of 8½-in. by 11-in. paper with a 6½-in. by 10½-in. invisible frame. Small tables should be grouped within this frame. Specific reference and explanation should be made in the text for each table.

7. Illustrations should be drawn in black ink on one side of 8½-in. by 11-in. paper within an invisible frame that measures 6½-in. by 10½-in.; the caption should also be included within the frame. Because illustrations will be reduced to 69% of the original size, the capital letters should be ⅛-in. high. Photographs should be submitted as glossy prints in a size that is less than 6½-in. by 10½-in. Explanations and descriptions should be made within the text for each illustration.

8. Papers should average about 12,000 words in length and should be no longer than 18,000 words. As an approximation, each full page of typed text, table, or illustration is the equivalent of 300 words.

Further information concerning the preparation of technical papers is contained in the "Technical Publications Handbook" which can be obtained from the Society.

This Journal is published bi-monthly by the American Society of Civil Engineers. Publication office is at 2500 South State Street, Ann Arbor, Michigan. Editorial and General Offices are at 33 West 39 Street, New York 18, New York. $4.00 of a member's dues are applied as a subscription to this Journal. Second-class postage paid at Ann Arbor, Michigan.

Subject and author indexes, with abstracts, are published at the end of each year for the Proceedings of ASCE. The index for 1958 was published as Proc. Paper 1891; indexes for previous years are also available.

Journal of the

SANITARY ENGINEERING DIVISION

Proceedings of the American Society of Civil Engineers

CONTENTS

September, 1959

Papers

s
g

Journal of the
SANITARY ENGINEERING DIVISION
Proceedings of the American Society of Civil Engineers

SEWAGE SLUDGE DISPOSAL IN WESTCHESTER COUNTY

Guy E. Griffin,[1] F., ASCE

ABSTRACT

The paper discusses the quantity and costs of sewage sludge disposal for several of the Westchester County plants. The variations are sufficient to give excellent comparisons of the processes.

INTRODUCTION

Westchester County, just north of New York City, covers an area of 457 square miles and is bounded completely on the west side by the Hudson River and on the east by Long Island Sound and the State of Connecticut. About eighty per cent of the County's 640,000 population live within thirty per cent of its area and are served by 130 miles of County-owned and operated trunk sewers in ten sewer districts. The trunk sewers discharge to five County sewage treatment plants. Three of these plants have outlets into Long Island Sound and two into the Hudson River.

Two of the Long Island Sound plants furnish only fine screening and chlorination before discharge into outfalls in thirty to forty feet of water. The third plant, located at New Rochelle, furnishes primary treatment consisting of grit removal, sedimentation, chlorination, separage sludge digestion, sludge dewatering on a vacuum filter, flash drying and incineration.

Both of the Hudson River plants now furnish only fine screening and chlorination, but construction is under way on a joint plant to replace their sewage treatment facilities and provide primary treatment as at New Rochelle, but with different details and with disposal of sludge by barging to seal after the volume has been reduced by sludge thickening and high rate digestion.

A comparison will now be made between sludge disposal at the plant in New Rochelle and that planned for the Joint Plant in Yonkers.

Note: Discussion open until February 1, 1960. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. Paper 2161 is part of the copyrighted Journal of the Sanitary Engineering Division, Proceedings of the American Society of Civil Engineers, Vol. 85, No. SA 5, September, 1959.

1. Deputy Commissioner of Public Works County Office Bldg., White Plains, N. Y.

New Rochelle

This plant is designed for 15 m.g.d., average dry weather flow, from a population of 81,000 in the year 1975, located mostly in Larchmont, New Rochelle and part of Pelham.

Design allowances for treatment and disposal of solids were as follows:

Suspended solids in raw sewage	0.2 lbs./cap./day
Removal by sedimentation	60%
Volatile Content	80%
Reduction of Volatile Solids	60%

Four fixed-cover, 65-foot diameter digesters with 20 ft. side water depth provide 3 c.f. capacity per capita. This is in accord with the Ten State Standards. Piping is so arranged that the four tanks may be operated as primary, or in pairs as primary and secondary units. Operating experience to date has found the latter arrangement to give better results. The average solids loading, computed from the above figures, amounts to 1.2 lbs./c.f./mc. For the maximum month taken at 125% of the average, the loading would be 1.5 lbs./c.f./mc.

Further computation gives the average daily yield of digested sludge for disposal as 5,054 lbs., or 2.5 tons, equivalent to 925 tons per year.

Disposal facilities consist of one 8 ft. by 8 ft. Komline Sanderson coil spring vacuum filter, having 200 sq. ft. of surface area or a filtering capacity of 1,000 lbs./hr. at the rate of 5 lbs./sq.ft./hr. on a dry solids basis, and one C. E. Raymond flash drying and incinerating unit of equal capacity. These units are housed in a separate building in which space, floor openings and foundations have been provided for duplicate units. The building is connected to the sludge digestion building by an access and pipe tunnel. Tanks were provided for elutriation of digested sludge but, to reduce construction costs, equipment for them was not provided and economic studies, based on operating experience and laboratory tests, do not warrant purchase and installation of this equipment at the present time. Also deleted from the original design were provisions for compressed sludge gas storage, which might have been utilized for sludge drying and burning during summer months, and provisions for storage and packaging of dried sludge. These provisions may be added later if they become desirable from an economic standpoint. Ash from the incinerator is pumped as a slurry to a lagoon.

The capital cost of the sludge disposal facilities, not including digestion tanks, was:

Building and Equipment, including pipe tunnel, as installed	$640,000.
Estimated cost for future second filter and incinerator	197,000
Estimated Total Capital Cost as designed	$837,000

The average annual cost as built would be:

Average Interest - $640,000 @ 3% by 0.5	$ 9,600.
Depreciation - 30 yr. life	21,300.
Annual Capital Cost	$30,900.

Unit Capital Cost as built per ton dry solids

$30,900 divided by 925 =	33.40

As designed with two filters and two incinerators, it is estimated that by working two shifts, four days per week, the facilities could handle 3,000 tons dry solids per year. On this basis the costs would be

Average Interest - $837,000 @ 3% by 0.5	$12,555.
Depreciation - 30 yr. Life	27,900.
Annual Capital Cost	$40,455.

Unit Capital Cost - as designed for full capacity -

per ton dry solids - $40,455 divided by 3,000 = 13.50

Economic studies are now under way for possible conversion of sludge digestion facilities to high rate, and use of the full capacity of sludge disposal facilities in connection with plans for better treatment of Rye and Mamaroneck sewage, which now receives only fine screening and chlorination.

New Rochelle

Filtration costs from operating records - per ton of dry solids:

Ferric Chloride	3.5% @ $.075/lb.	$ 5.25
Lime	10% @ $.011/lb.	2.20
Water	300 c.f. @ $.18/100 c.f.	.54
Power	90 KWH @ $.02/KWH	1.80
Labor	2.6 hrs. @ $2.25/hr.	5.85
Filtration Cost - Design Capacity		$15.64

Labor cost is based on six hours operation with two hours for start-up and clean-up. Filtration costs, when the use of two filters is required, would be reduced somewhat, as one man could operate both filters, making the labor cost per ton $ 2.92

Thus, the filtration cost for maximum capacity with two filters becomes - per ton dry solids $12.72

New Rochelle - Incineration costs per ton dry solids:

Power	152 KWH @ $.02/KWH	$ 3.04
Water	350 c.f. @ $.18/100 c.f.	.63
Settled Sewage	200 c.f. @ $.05/100 c.f.	.10
Auxiliary Fuel	27.5 gal. No. 2 oil @$.122/gal.	3.36

Labor	2.6 hrs. @ 2.25/hr.	5.85
		$12.98

Incineration costs with two incinerators, when the use of both is required, would also be reduced as only one attendant would be necessary, and would become - per ton $10.06

SUMMARY

New Rochelle Sludge Disposal Costs

	Design Flow Basis	Max. Capacity Duplicate Units
Tons dry solids/year	925	3,000
Capital Cost) Interest & Depreciation) 30 yr. life)	$33.40	$13.50
Operating Cost		
Filtration	$15.64	$12.72
Incineration	12.98	10.06
Total per ton dry solids	$62.02	$36.28

Yonkers

The new joint treatment plant at Yonkers is designed for an average annual flow in 1980 of 63 m.g.d. from a population of 505,000. Treatment units will consist of aerated grit and inlet channels, two-hour sedimentation and chlorination. Effluent discharge will be through a short outlet into water about 40 feet deep. Sludge treatment and disposal facilities to be provided consist of two sludge thickening tanks, each 45 ft. in diameter with 9 ft. side water depth and 5 ft. deep cone bottom; two fixed-cover sludge digestion tanks each 70 ft. in diameter with 30 ft. side water depth; two fixed-cover sludge storage tanks, each 56.5 ft. in diameter with side water depth of 23 ft.; a 200 ft. docking space and a sludge barge of 1600 ton capacity to be towed to sea. The barge, as designed, is to have separate hoppers for grit with steeply sloping sides and large bottom gates. The grit will be loaded by pneumatic ejectors.

Provision has been made in plant layout and design for the future addition, when required, for sixty-minute aeration with 10% return sludge, additional sedimentation, sludge thickening and digestion facilities. It is estimated that such facilities would provide for 80% removal of suspended solids from 92 m.g.d. in the year 2010.

Basic Allowances - Yonkers

Design Year	1980
Population	505,000
Annual Average Sewage Flow, m.g.d.	63
Suspended Solids in Raw Sewage - p.p.m.	154
lbs./cap.	0.16
Removal by Sedimentation, per cent	60
Annual Average lbs./24 hrs.	48,500
lbs./mo.	1,455,000
Maximum Mo. lbs. per month.	1,839,000
Dry Solids - Tons per year	8,850

Recommended operation calls for continuous pumping of sludge to the sludge thickening tanks which are designed to operate at 800 gal./sq. ft., and thicken the sludge to approximately 10% solids. With one thickener in operation, the solids loading will be 30.4 and, for two tanks, 15.2 lbs./sq. ft. Provisions are made for adding settled sewage to maintain the overflow rate. Thickened sludge is to be pumped continuously to the digesters and digested sludge transferred continuously to the sludge storage tanks.

The digesters are to be provided with draft tube mixers to be operated continuously to maintain a homogeneous mixture of raw and digesting sludge. Sludge gas will be utilized for heating and for the operation of two direct connected gas engine driven blowers.

Solids loading of the digesters in 1980 is expected to be:

Annual Average	6.2 lb./c.f./mo.
Maximum Month	7.8 lb./c.f./mo.
It is estimated that in the year 2010, the average loading would become	9.0 lb./c.f./mo.
and if aeration should be added it might become	12.0 lb./c.f./mo.

On a volume basis, the sludge to the digesters at 10 per cent solids will amount to 7,240 c.f. per 24 hrs., giving 32.6 day displacement. The sludge digester volume per capita is 0.47 in 1980 and 0.33 c.f. per person in the year 2010.

The two sludge storage tanks with a combined capacity of 120,000 c.f. will afford 16.6 days storage for the digested sludge at the 1980 design basis.

The 1600 ton capacity sludge barge will have a volume capacity of 50,000 cu. ft. This is a 6.9 day production capacity. Thus, it will be necessary to make a trip to sea every seven days when the 1980 allowances are reached. For the estimated 1960 population of 400,000, this capacity barge will need to make a trip about every nine days. The barge which has been designed for this work will be 201' 9" long by 36' beam, with a loaded draft of 10' 9".

The estimated capital costs of the sludge disposal facilities, not including sludge thickening, digestion or storage tanks which are really sludge treatment units, are as follows:

Dock and related work, including removal of part of the existing outlets and provision of a

bulkhead (completed)	$245,000
Estimated Barge Cost	300,000
Total Capital Cost	$545,000

The average annual cost, as designed, would be:

Average Interest - $545,000 @ 3% by 0.5	$8,180.
Depreciation - 30 yr. life	18,200.
	$26,380
Unit Capital Cost per ton Dry Solids $26,380 divided by 8,850	$2.98

From preliminary investigations, it is believed that the County can receive bids not to exceed $53,000. as an annual fee for operation, management and maintenance of a County-owned barge, and it is further estimated that towing charges will be approximately $680. per trip.

On this basis:

Operation, management and maintenance per ton dry solids - $53,000 divided by 8,850 =	$5.99
Towing with sludge at 6% solids 1600 by .06 = 96 tons dry solids per trip Per ton dry solids - $680 divided by 96 =	7.08
Estimated Total Operating Costs	$13.07

Estimated Total Capital and Operating Cost per ton Dry Solids $16.05

The following table shows a comparison between the unit costs estimated for sludge disposal, exclusive of sludge treatment, at New Rochelle and Yonkers:

Summary of Estimated Unit Costs for Sludge Disposal

	New Rochelle		Yonkers
	Design Flow Basis	Max. Disposal Capacity-2 Units	1980 Design Year
Dry Solids - Tons/yr.	925	3,000	8,850
Capital Cost - 30 yr. Life per ton dry solids	$33.40	$13.50	$2.98
Operating Costs.			
Filtration - per ton dry solids	15.64	12.72	
Incineration " " " "	12.98	10.06	
Barge - per ton dry solids			5.99
Towing- " " " "			7.08
Total Cost Per Ton Dry Solids	$62.02	$36.28	$16.05

It is interesting to note in this comparison that the unit cost of sludge disposal at New Rochelle could be drastically reduced by loading to capacity the facilities as originally designed. This may be possible in the future, as already mentioned. A further reduction in these costs at such time could perhaps be realized by providing the equipment necessary for compressing and storing sludge gas to be used as auxiliary fuel. A still further cost reduction might be realized if a market could be developed for the sale of heat dried sludge at a price in excess of the additional capital and operating costs for drying, bagging and marketing.

It is fairly evident, however, that with all these possible reductions the unit cost would still be appreciably higher than that estimated for barging to sea from the Yonkers Plant. It is also a fair assumption that, had barging to sea been recommended for New Rochelle, there would have been much opposition from small boat owners and residents of the high class residential area nearly surrounding the harbor. Cost of dredging a necessary channel and development of docking facilities would have been high because of rock, and towing charges would probably be more because of a somewhat greater distance and more difficult course to the dumping grounds.

Therefore, it is concluded that sludge disposal by barging to sea is economical in the cases considered above but may not be feasible for all sewage treatment plants in coastal cities.

Journal of the

SANITARY ENGINEERING DIVISION

Proceedings of the American Society of Civil Engineers

FILTER PLANT DESIGN [a]

Richard Hazen,[1] F. ASCE

ABSTRACT

The recent design of several filter plants ranging in size up to 200 mgd for municipal and industrial purposes has provided opportunity for critical analysis of laboratory data and actual plant performance. Particular attention was directed towards design criteria for mixing, flocculation and sedimentation, and the arrangement of filters and filter piping. Decisions influenced by local factors are reviewed and major differences in old and new plants described.

This paper deals with several features of filter plant design selected chiefly to illustrate modifications and departures from conventional practice. They have resulted from special investigations over the past three years in connection with the design of a number of plants. The work has included the design of a 200 mgd addition to the Springwells plant in Detroit, the design of a 20 mgd addition to an industrial water plant in Maryland, two smaller plants for municipal service, and the functional design of a 150 mgd plant for Wayne County, Michigan. The design reflects not only studies of the particular plants involved, but also a critical inspection and analysis of eleven United States water plants during the summer of 1956.

The design material presented is distinctly current, and some of it cannot be supported by operating data at this time. The Springwells addition is under construction and will be in operation by the summer of 1959. One of the smaller plants has been in operation a few months. The others are scheduled for construction in the near future.

Mixing, Flocculation and Sedimentation

The importance of good pretreatment ahead of rapid sand filtration is generally accepted. It has been demonstrated many times by the miseries of

Note: Discussion open until February 1, 1960. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. Paper 2161 is part of the copyrighted Journal of the Sanitary Engineering Division, Proceedings of the American Society of Civil Engineers, Vol. 85, No. SA 5, September, 1959.

a. Presented at the October 1958 ASCE Convention in New York, N. Y.

1. Partner, Hazen and Sawyer, New York, N. Y.

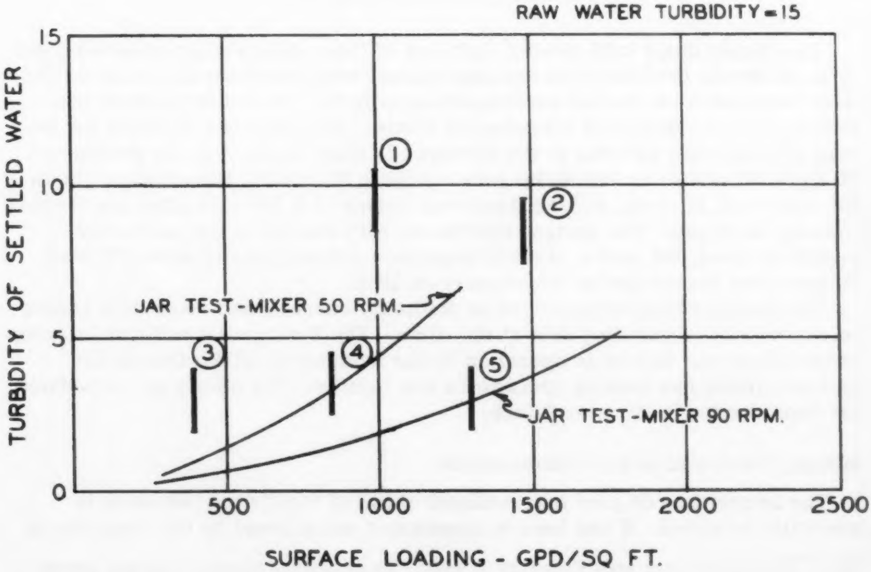
filtering improperly treated water and by the almost trouble-free operation obtained with good coagulation and sedimentation.

In plants with good flocculation the customary "rapid mix" may be unnecessary. A number of water plants rely entirely on the turbulence through raw water pumps and conduits for dispersing chemicals. In one large plant on the Great Lakes the floc forms before the water reaches the mixer, and the mechanical rapid mix equipment has been found to impair flocculation. In our recent designs, we have included a mixer in the larger plants but not in the smaller plants. Sometimes it is easier to provide a mixer than to deliver and feed chemicals ahead of the raw water pumps.

Laboratory tests of flocculation and sedimentation of Detroit River water have confirmed and extended the theories on flocculation presented in recent years by Camp, Stein and others. High-speed flocculation causes more of the suspended matter to be entrapped than slow-speed flocculation, and the floc particles produced, though relatively small, settle at reasonable velocities. Slower flocculation produces a mixture, some of which settles rapidly and some extremely slowly. Within limits, the proportion of turbidity removed varies with the energy transferred to the water during flocculation.

Some water treatment plants have little or no flocculation. Others have adequate flocculation, but the benefits are lost because of settling tank design and operation. The divergence in the results obtained at several Great Lakes plants is indicated in Exhibit 1.

The short vertical lines on Figure 1 show the results of flocculation and sedimentation for each plant. They were plotted from one year's operating



SETTLED WATER TURBIDITY-GREAT LAKES PLANTS

Fig. 1

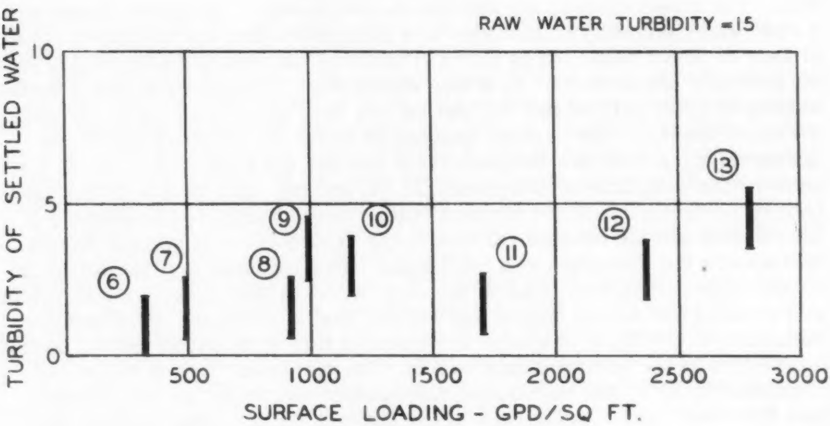
data. For direct comparison the results are based on the performance with a raw water turbidity of 15. This is a little more than the average turbidity of Detroit River water but is only a fraction of the peak turbidity of 150 ppm occasionally encountered. At Water Works Park (1) in Detroit with a surface loading of 1,000 gallons per day per square foot, the settled water turbidity averaged about 9. The surface loadings at Water Works Park and at the Springwells plant (2) are comparatively high but not such as to account for the unfavorable comparison with plants (3), (4) and (5). The causes of the unsatisfactory performance of the water at Water Works Park are the absence of flocculation and the nonuniform flow in the around-the-end settling basins. At Springwells the flocculation is inadequate. The Springwells plant design included eight mechanical mixers, but only two were installed. Conduit velocities between the mixing basins and settling basins are high. Furthermore, variations in the depth of sludge and times of flow have shown uneven distribution of flow in the sedimentation basins. The Northeast Station in Detroit, completed in 1956, has mechanical flocculation and performs much better than the older units. At the time of the surveys the Northeast Station was operating at low rates and the data were not suitable for inclusion in Figure 1.

The two curves in Figure 1 give the results of laboratory jar tests on Detroit River water. For the tests shown, the water was flocculated for 32 minutes with stirrer speeds of 50 and 90 rpm and was then allowed to settle quietly in the jar. Samples were taken 5 inches below the surface of the water after settling times ranging from 0 to 60 minutes to determine the effect of surface loading. These particular curves show principally the effects of flocculator speeds and obviously do not reflect variables inherent in plant hydraulics and operation. They do demonstrate, however, the importance of both good flocculation and reasonable surface loading in getting good performance out of the basins.

A preliminary study of turbidity removals at plants treating river waters in the eastern United States is shown in Figure 2. The data are on a basis similar to that used for comparing the Great Lakes plants, but the variations in performance are less. No coagulant aids were used. The good removals obtained by the three solids-contact units at high surface loadings (11, 12 and 13) are evidence of what can be accomplished in properly designed equipment of this type. The good performance apparently is due to the better flocculation in solids-contact reactors.

New Springwells Flocculators

The general arrangement of the new Springwells flocculators is shown in Figure 3. There are four basins, each 590 feet long and 102 feet wide. At the inlet end of each basin there are five parallel flocculation chambers, each chamber divided by baffles into four compartments to minimize short-circuiting. The flocculation time is 35 minutes at the plant's rated capacity of 200 million gallons a day. Each flocculator assembly consists of eight paddles mounted on a single longitudinal shaft with the drive in the dry gallery at the end of the basin. The first six paddles are identical with four 7-foot-long arms per paddle and four blades per arm. The last two paddles in the fourth compartment have only two arms, each carrying two blades. Laboratory studies indicated that optimum settling characteristics are obtained by a quick reduction in the last compartment rather than by a gradual reduction in flocculation speed. The flocculators are designed for a maximum peripheral



6-10 HORIZONTAL FLOW BASINS

11-13 SOLIDS CONTACT UNITS

SETTLED WATER TURBIDITY - RIVER PLANTS

Fig. 2

speed of 1.75 feet per second, with a 3 to 1, infinitely-variable speed drive.

The water passes from the last flocculation compartment through a perforated baffle wall into the settling compartment. The holes in the baffle are 12 inches in diameter, placed 3 feet center to center. One of the perforated baffle walls is shown in Figure 4.

Proposed Wayne County Flocculators

The proposed Wayne County flocculator design is shown in Figure 5. The general arrangement of flocculating and settling basins is similar to that at Springwells, except that in the Wayne County design, turbine-impeller flocculators on transverse horizontal shafts are proposed. There will be five shafts, each carrying six impellers. The turbine-impellers are to consist of a circular steel plate not more than 7 feet in diameter, on which are mounted 6 to 10 flat steel blades approximately 24 inches long by 12 inches wide. The impellers are to "pump" not less than 60 cfs, with a maximum peripheral speed of 2 fps. The impellers will have an infinitely variable speed drive with a range of 3 to 1. The five flocculating compartments are to be separated by transverse concrete baffle walls with 4' × 4' ports opposite each impeller. There are no longitudinal training walls. The flocculating time will be 40 minutes when the plant is operating at rated capacity.

The type of flocculator proposed is used successfully in solids-contact basins, and with vertical shafts, in conventional basins. In the writer's opinion, they are more efficient than the paddle type in that they avoid a relatively dead space along the axis of the paddle. In the standard paddle design, most of the flocculation takes place at the outer periphery. The turbine impellers exert a more uniform mixing action throughout the flocculating chamber.

Solids-Contact Units for Industrial Water Plant

Two 85-foot-square solids-contact units are proposed for a 20 mgd addition to an existing 16 mgd industrial water plant. Each new unit is rated at 11.5 mgd. Limited space and economy have been factors in selecting this type of equipment, but equally important has been our desire to overcome or dampen sudden changes in raw water quality. The raw water is low in color but subject to acid-mine drainage. Its characteristics are indicated by the following data:

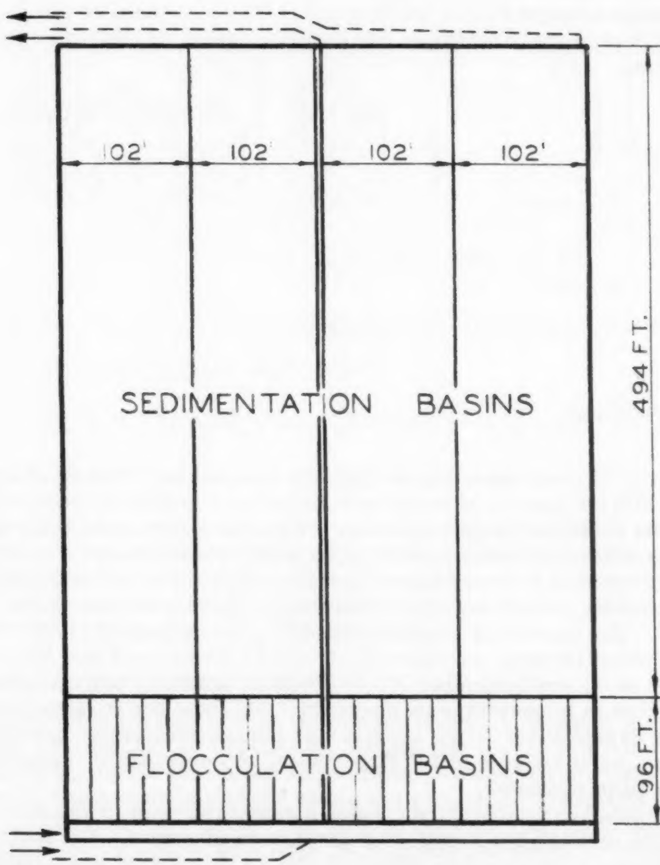
	Maximum	Average	Minimum
Turbidity	250	7	1
Iron, ppm	20	1.5	0.6
Manganese, ppm	1.2	0.3	0.05
Aluminum, ppm	16	3.5	0
Hardness, ppm as $CaCO_3$	150	50	20
pH	8	4-5	3
Temperature, °F.	90	65	32

The quality of river water varies with the seasons and river discharge and especially with the amount of water released from a regulating reservoir upstream. The reservoir is on a tributary stream free from acid-mine drainage, and the water impounded is cool, soft, and clear with a pH of about 7.

The North Branch Potomac River is shallow and the water temperature ordinarily follows closely the air temperature. Typical variations are shown in Figure 6. The curves for February 25, 1955, and August 17, 1956, show the diurnal effect of warm sunshine on the river. February 5 and May 2, 1956, were cloudy days, and September 1, 1956, was an unusually hot, sunny day. The sharp drop in temperature on August 18, 1955, was due to opening of the gates at the Savage River Dam. On this date the temperature dropped 18°F. in five hours, or at the rate of 3.6°F. per hour. A temperature change of 2°F. per hour is quite common.

A rapid change in temperature creates thermal currents in any settling basin, and the existing horizontal-flow basin is no exception. For temperature rises, the same effect should be expected with an upflow type basin unless provision is made for the recirculation of large volumes of sludge through the reaction zone. This results in a high solids contant in the reaction zone, which can offset thermal effects. The specifications for the new units require satisfactory operation with a settleable solids content in the reaction zone exceeding 5% and require high-rate recirculation through approximately 1/3 of the volume of each unit. Total circulation will be up to three times the throughput. Equipment manufacturers have indicated their ability to meet the specified performance which limits the settled water turbidity to not more than 3 ppm.

In choosing between solids-contact units and horizontal settling basins, a number of factors are usually involved. One that has discouraged solids-contact units in two instances has been the stipulation that three- or four-hour



NEW SEDIMENTATION AND

SPRINGWELLS PLANT,

Fig. 3

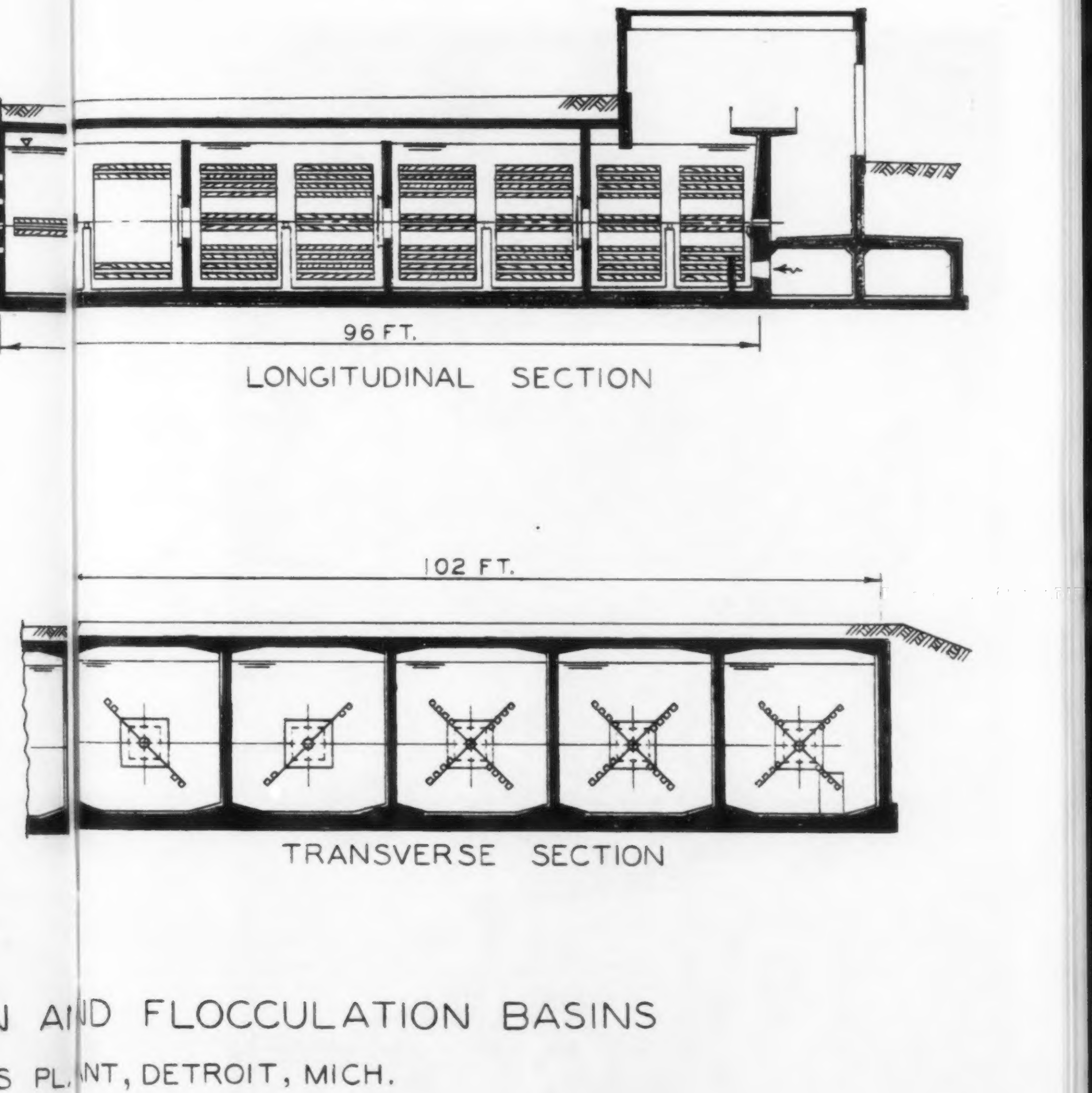

96 FT.
LONGITUDINAL SECTION
102 FT.
TRANSVERSE SECTION
AND FLOCCULATION BASINS
PLANT, DETROIT, MICH.

Fig. 3

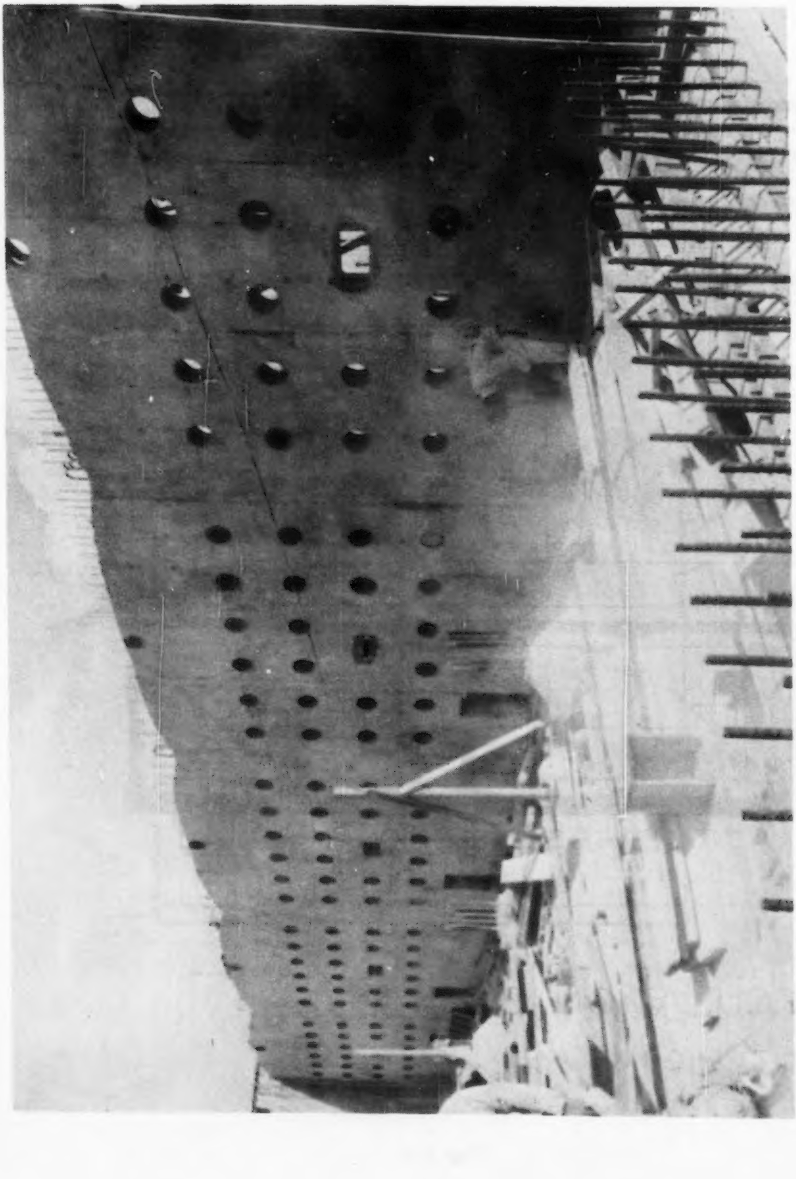
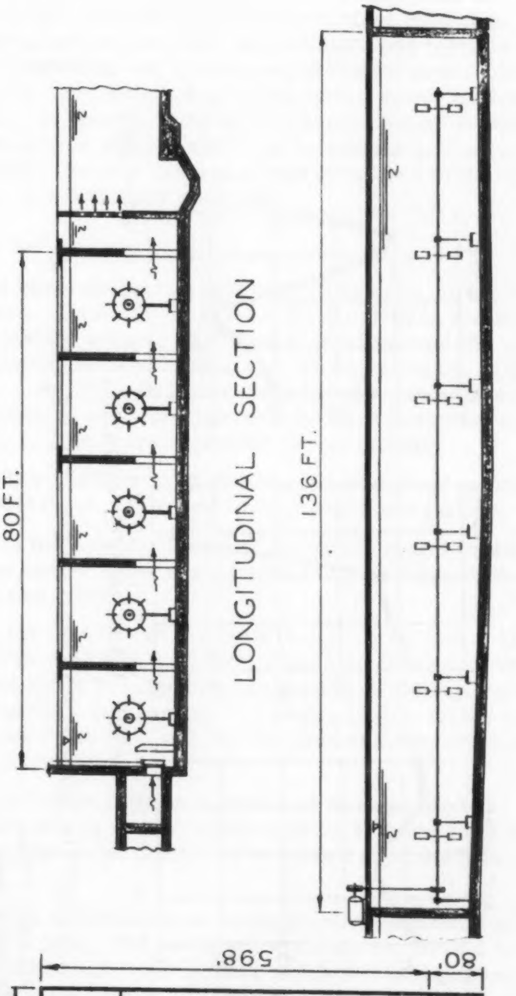
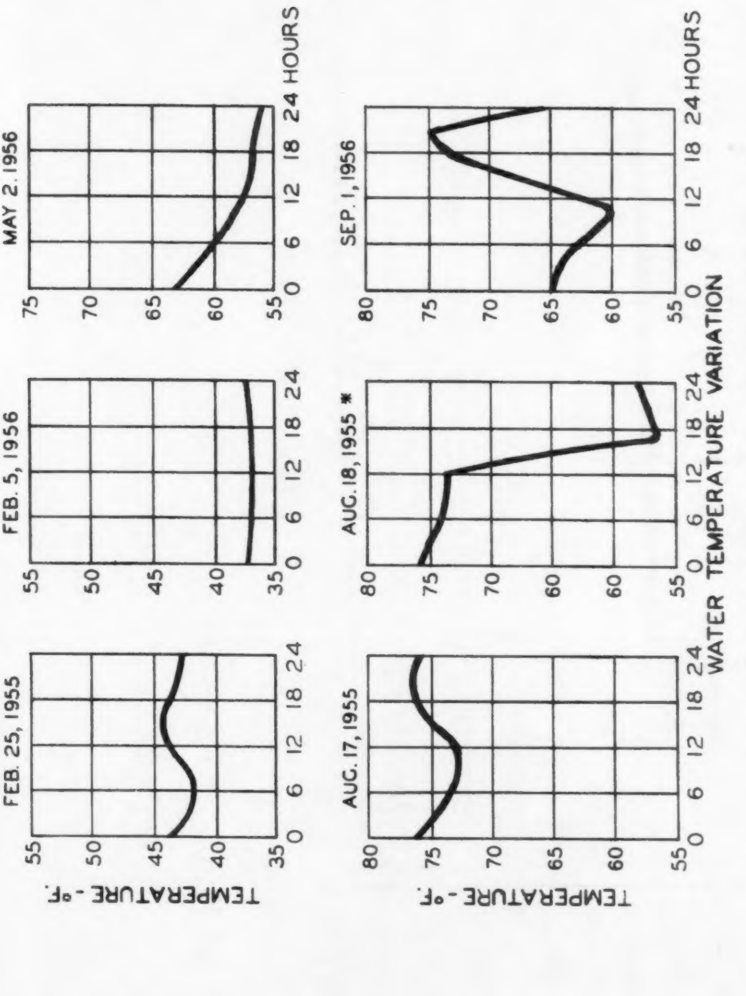


Fig. 4



PROPOSED SEDIMENTATION AND FLOCCULATION BASINS
WAYNE COUNTY PLANT

Fig. 5



NORTH BRANCH POTOMAC RIVER AT LUKE, MARYLAND.

* WATER RELEASE FROM SAVAGE RIVER DAM

Fig. 6

settling time be provided, not for sedimentation, but rather to assure ample time for prechlorination and taste-control measures. In such cases the solids-contact basins offer no advantage and there is little reason to incur the cost of installing and maintaining the necessary equipment. (An important exception is the treatment of highly turbid water in which a solids-contact unit affords easy sludge removal.) A second situation which has raised some doubts and for which there is no immediate answer involves intermittent operation. Several problems have developed in the operation of a small plant used only a few hours each day.

Filter Beds

The basic design of rapid sand filters has changed little in the past 40 or 50 years. Higher filter rates are permitted in some states, coarser sand is used, and the surface wash has attained recognition as a worth while and sometimes necessary auxiliary. In designing the Springwells addition, some features not found in the existing Detroit plants have been introduced. These are indicated by the sketches of the old and new Springwells filter layout in Figure 7. The more important are as follows:

1) The new filter building had to be long and narrow to fit the available land, and there was room for only one filter gallery.

2) In the new works settled water is brought to the outer end of the filters, and the center gallery is reserved for the filtered water headers, wash-water piping and controls.

3) The new filters are built in units of 57 feet × 19 feet, compared to dimentions of 40 feet × 27 feet in the old. The narrower filters are more economical to build and permit the use of short, precast-concrete, wash-water gutters discharging to a center gullet. In the old filters the gutters were nearly 40 feet long, of cast iron and supported at a number of points from crosswalks.

4) In the original plant each bed was operated and controlled separately. The new filters are arranged in pairs with one inlet and one rate controller serving two beds. Each bed is washed separately to avoid excess wash-water rates.

5) The underdrains in the original plant consist of headers and perforated cast iron pipe. The new plant will have perforated vitrified tile blocks (Leopold) except for one filter which will be equipped with perforated precast concrete blocks (Criscrete).

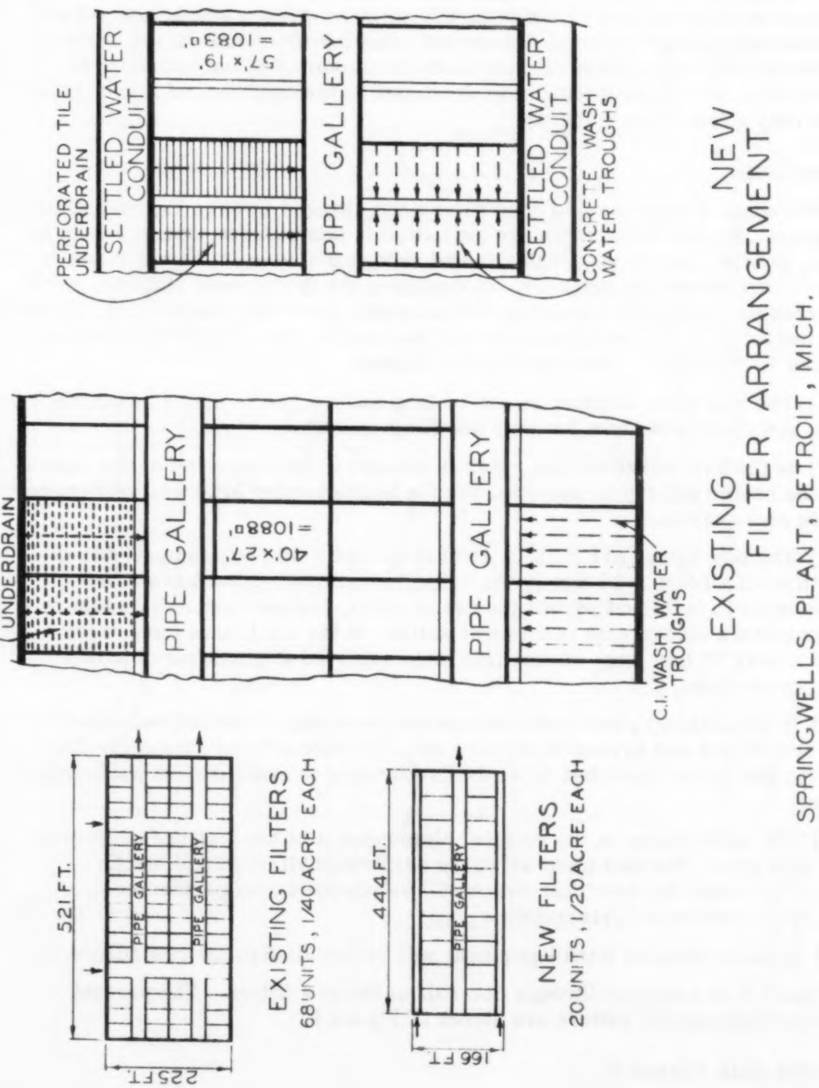
6) Rotating surface wash equipment will be installed in the new filters.

Figure 8 is a section through one-half of the new filters. The precast concrete wash-water gutters are shown in Figure 9.

Variable Rate Filtration

For some time Mr. H. E. Hudson, Jr.,* has been studying the effects of filtration rates on the floc penetration of sand beds and the quality of water

*M. ASCE; Partner, Hazen and Sawyer, Engineers.



EXISTING NEW
FILTER ARRANGEMENT
SPRINGWELLS PLANT, DETROIT, MICH.

Fig. 7

filtration rates on the floc penetration of sand beds and the quality of water produced. Theoretical considerations have led to the conclusion that the best results (that is, highest removal of turbidity and bacteria) are obtained if the filters are operated at high rates when they are clean and at lower rates when they have become dirty. In a conventionally operated filter, the rate controller induces head loss in the filtered water piping at the start of a filter run, which is gradually reduced as the filter becomes clogged, so that the filtration rate remains constant throughout the run. In a number of plants the rate controllers do not function because of neglect, and in some others the throttling mechanism has been removed to increase the capacity of the filters. In such instances the loss-in-head through the filter beds is relatively constant, and the discharge from each filter declines as the bed becomes dirty. These plants are operating as uncontrolled variable-rate filters.

To demonstrate the possibilities of variable rate filtration, two filters in the Wyandotte, Michigan, plant have been operated for more than a year—one with a conventional rate controller and the other with a fixed orifice sized to limit the maximum flow through the plant to approximately 1-1/2 times the average flow. A full report on this experiment is beyond the scope of this paper. However, it can be stated that both filters have turned out consistently good water. Throughout the tests, the variable-rate unit has had the edge in respect to the quantity of water filtered, the wash water required, and the turbidity of the final effluent. The results have been so satisfactory that orifice controls will be substituted for rate controllers in the 20 mgd addition to the industrial water plant. The elimination of rate controllers makes it desirable to reconsider the hydraulics between filters and clear well and introduces some changes in operating procedure. However, variable-rate filtration shows real promise. It should simplify construction and reduce the cost of filter plants.

Clear Well or Filtered Water Storage

Detroit has long employed a "shunt system" by which filtered water goes directly from the filters to the high lift pumps with excess water spilled over a weir into the filtered water storage reservoirs. At times of peak demand, the pumps draw on both the filters and the reservoirs. This arrangement reduces the high lift pumping head by several feet—enough to effect a substantial saving in power costs over the years. The shunt system is good, as long as detention in the clear well is not needed to complete chemical reactions and circulation through the reservoirs is not necessary to keep the water fresh and palatable.

The filtered water storage for the proposed Wayne County plant will consist of three 10 mg steel tanks, 15 feet deep, with the overflow some 7 feet above the water level on the filters. The filtered water headers discharge into four equalizing basins at the pumping station from which the water may be routed to the high lift pumps either directly or by way of the storage reservoirs. The initial installation will include six variable-speed low-head transfer pumps for lifting the water to the storage reservoirs. Two of the six units are arranged to serve as both transfer and wash-water pumps.

The use of transfer pumps between the filters and clear well in the Wayne County design is dictated largely by foundation conditions. The site is underlain by a 10-foot layer of firm clay at the surface and 60 feet of soft clay that will not support concentrated loads. The low lift pumping station caisson

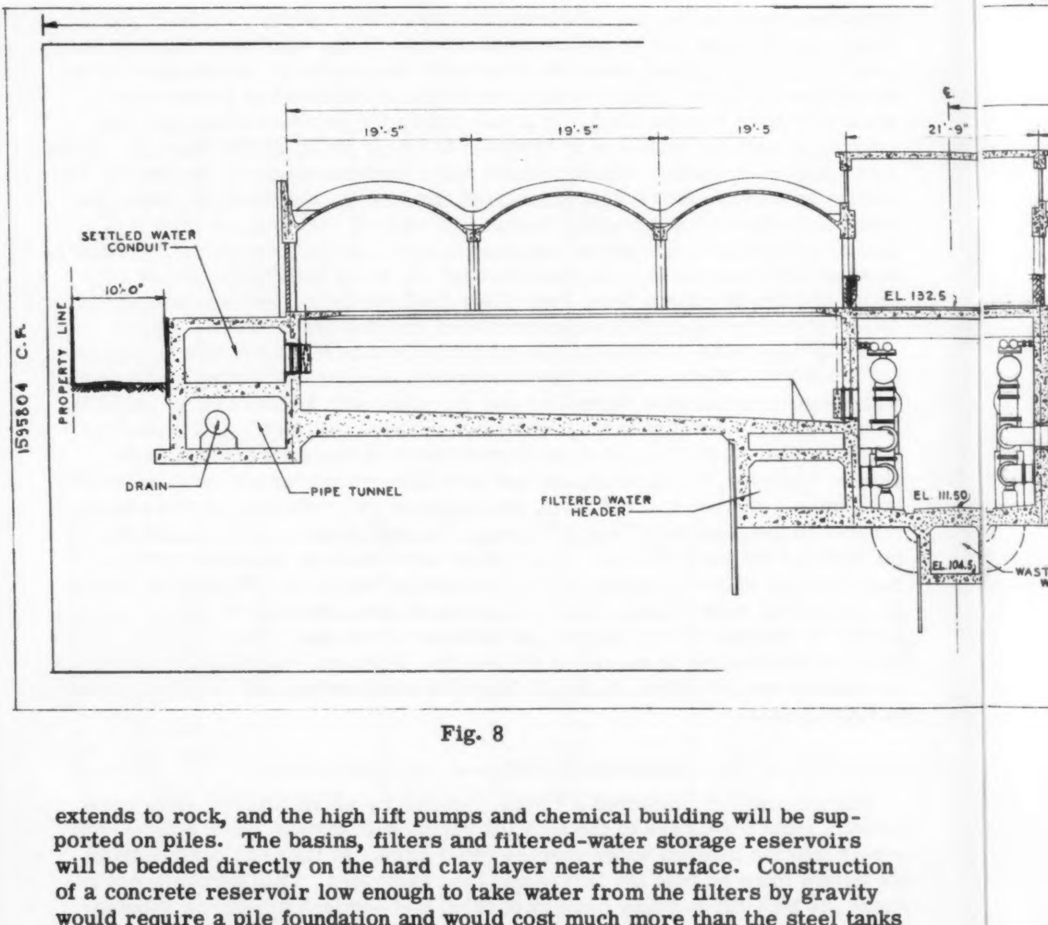


Fig. 8

extends to rock, and the high lift pumps and chemical building will be supported on piles. The basins, filters and filtered-water storage reservoirs will be bedded directly on the hard clay layer near the surface. Construction of a concrete reservoir low enough to take water from the filters by gravity would require a pile foundation and would cost much more than the steel tanks and transfer pumps proposed. Furthermore, the high clear well elevation eliminates the usual loss in pumping head when the reservoirs are not full.

Transfer pumps have been used at the new plant in Holland, Michigan, and at a few other plants. There is no clear well at all in the South Plant of the St. Louis County Water Co., and the water is pumped directly from the filters to the distribution system and storage reservoirs on the system. There is much to be said for this double use of storage capacity. However, it is undesirable to connect pumps directly to filter headers because unavoidable pump surges may force floc through the filters. A pump suction chamber open to the atmosphere and large enough to take the surges prevents this difficulty.

Superstructure

The new Northeast Station in Detroit has thin-shelled concrete vaults over the filters running perpendicular to the filter gallery. As shown in Figure 8, the roof over the new filter beds at Springwells consists of three thin-shelled

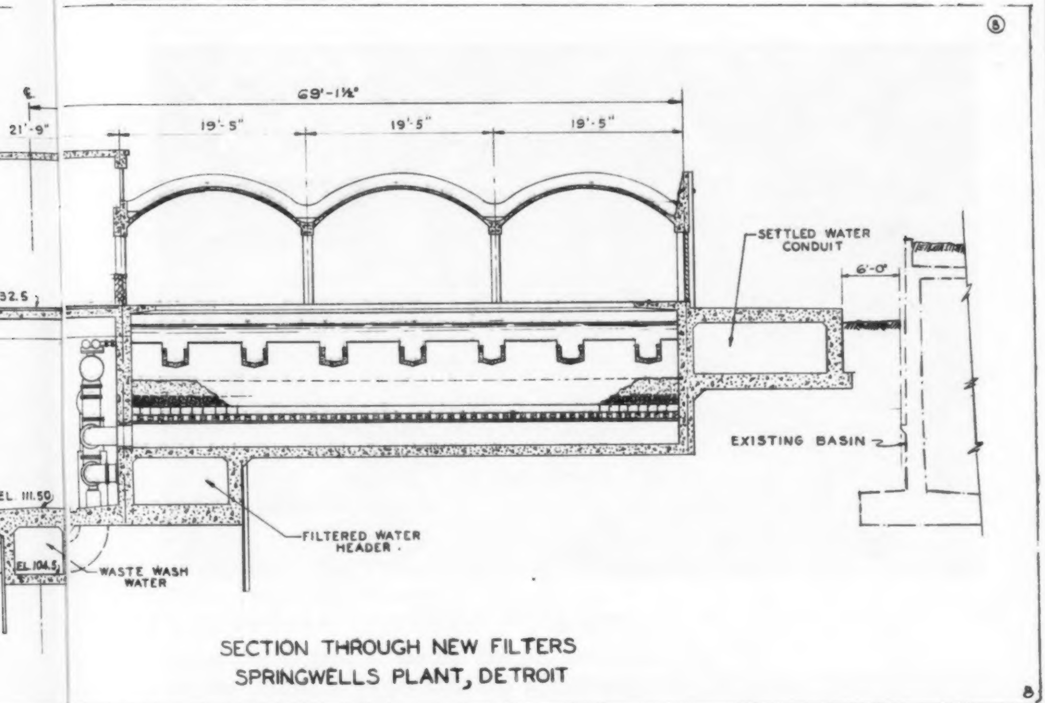


Fig. 8

vaults, with the barrels running the length of the building. The shells have a 19'-5" chord and unsupported length of 44 feet, and rest on columns mounted on the division walls between filters. The roof over each bank of filters was poured in three sections, two 132 feet long and one 176 feet long. The forms were dropped after the first section was completed and slid along to the next section for re-use. The short-chord shells provided a smooth concrete ceiling, easy to clean and free from maintenance problems, and avoided heavy concentrations of foundation loads. The last was important at Springwells, where practically all of the new structures have been built on mats without piles. Approximately 1,500,000 lineal feet of piles were driven under the original station. Figure 10 shows part of the roof under construction.

CONCLUSION

This brief review of a few selected items hardly leads to specific conclusions. The design features described are the result of evolution rather than new discovery, and the valuable experimental work at Detroit and other cities in years past is fully acknowledged. Application of these studies, adequate attention to basic principles, and critical detailed design are the surest roads to lower costs and better performance.

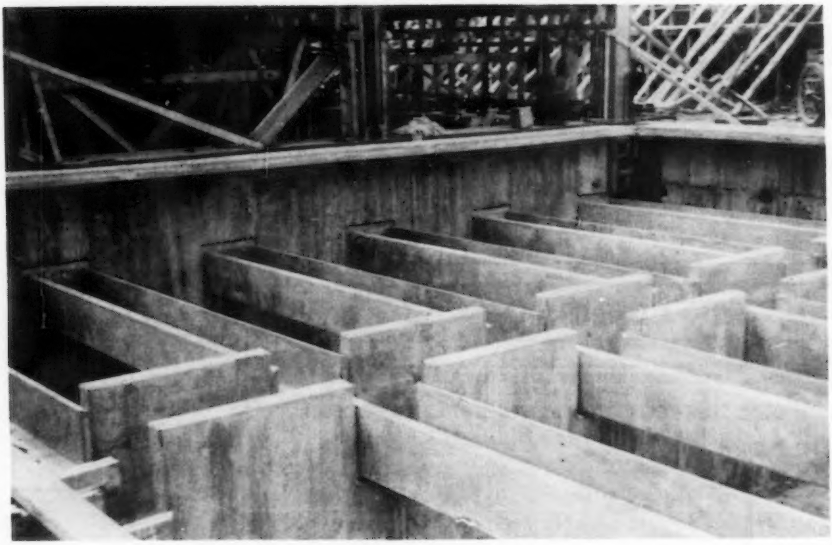


Fig. 9

Fig. 10

Journal of the

SANITARY ENGINEERING DIVISION

Proceedings of the American Society of Civil Engineers

CONTENTS

DISCUSSION

RESISTANCE OF SEWAGE SLUDGE TO FLOW IN PIPES[a]

Closure by Tsung-Lien Chou

TSUNG-LIEN CHOU,[1] F. ASCE.—The writer is very much obliged to those who have taken deep interests in the discussion, either through this Publication or by correspondence and have made valuable contributions to the subject in hand.

Drs. Li and Whitington and Mr. McPherson made some comments on the dimensions of the coefficients of rigidity and viscosity. It should be pointed out that "pound" in both of these cases is understood to mean mass which is related to force or weight as follows:

One pound force = one pound mass x g (the acceleration due to gravity)

Reasoning in this way, all equations will be found homogeneous and consistent. Nevertheless, the writer does appreciate Dr. Li's complete illustration to warn off the common pitfall of "pound mass" and "pound force" in the so-called Engineering Units of Ft - Lb - Sec System (and of C - G - S System also).

As to the nature of sludge flow, Dr. Li produced a fine suggestive outline along the general contour of rheology and Professor Babbit summarised the whole field in practical and concise terms. Regarding the heterogeneous characteristics, it has been known that, neither μ in suspension flow nor η in plastic flow is constant; thixotropy is also an important property of sludge; besides, Brownion movement and flocculation have their shares in influencing the flow. About the moisture, not only the amount, but also the state is an important factor in determining the physical properties of a sludge. A good illustration is the recent personal experience of Colonel A. B. Morrill, Fellow, ASCE, who states in his letter "The treatment plant at Caroro got into trouble and they sent us a sample of river water that contained 14% by weight of suspended solids. After two hours of settling in an Imhoff cone the mud at the bottom contained about 40% of solids. The Venezuelan engineer had the idea to try to siphon the mud out of the cone with a 1/4 inch rubber hose. Much to my surprise it ran almost like water under a head of only a foot or two. Any sewage sludge with only 60% moisture is practically solid. It must be a matter of fine particles and the lack of any fibres in the solids in the water." "On the other hand, a brilliant sanitary engineer told the writer not long ago that, according to his twenty years' experience in operating and testing treatment plants, any moisture content less than 88% will block up pipes completely." From these two cases cited above, it can easily be inferred that, a

---

a. Proc. Paper 1780, September, 1958, by Tsung-Lien Chou.

1. Engr. of Hydraulics and Hydr. Structures, Clinton Bogert Engineers, New York, N. Y.

high proportion of sludge moisture is held in captivation in one way or other, so that the amount of free water available for flow is much smaller than the total.

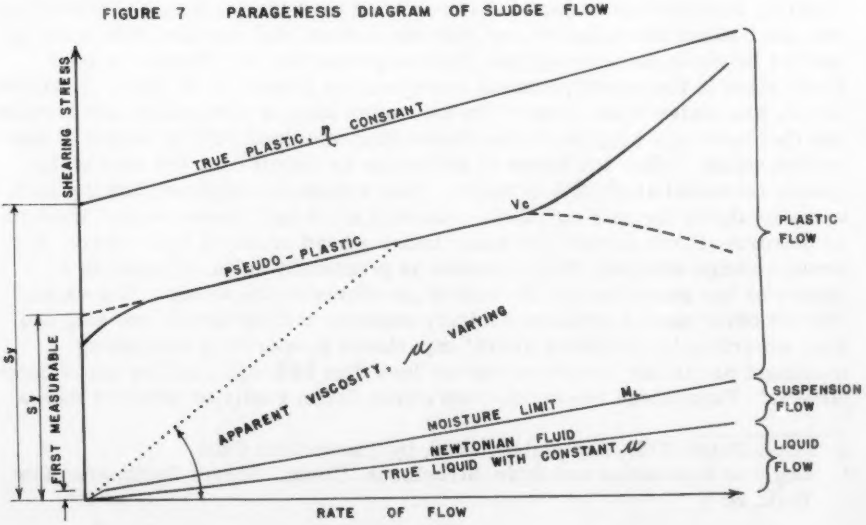
According to some recent experiments,[21] the fibre in suspension can produce yield stress and plug flow even at high moisture contents of 99.25% to 99.50%. It is found that, in laminar stage, the head loss per unit length of pipe is a function of the pipe circumference. That shows the resistance is localized to the laminar sheer in a narrow zone near the bundary.

To deal with flow of such variable physical properties, the essential conception of flow characteristics is that it may flow either as a liquid or as a plastic, as emphasized by Professor Babbit. Perhaps a general view may be expressed in the paragenesis diagram (Fig. 7) showing the relations but not the actual magnitudes of the various kinds of flow. The non-Newtonian flow of the sewage sludge lies in a region with the true plastic as the upper boundary and true liquid as the lower one. The line of moisture limit M_L divides the whole region into two main zones, one of plastic flow and the other of flow in suspension. The flow in lower zone is governed by Eqs. (10), (11) and (12).

Regarding Eq. (9), Mr. R. C. Worst of the British Hydrodynamics Research Association (by letter) considered that the increase in head loss (over true liquid) is presumably increased merely as the pipe friction coefficient is increased by the rise in viscosity. Of course, that is just another way of expressing the same effect without modifying the actual quantity, if it is realized the increase in g is hardly more than four per cent in the ordinary range of suspension flow.[22]

In plastic flow, the sewage sludge is classified as pseudo-plastic. Under laminar state, the curve may straighten up at higher velocities, so that an apparent viscosity may be approximated by Eq. (2). What will happen beyond the critical velocity v_c? Former experiments showed increase in sheering stress for the ordinary sludges.[1,2] But the experiments on fibre suspension shows the other way. Therefore, it is better to limit the field of application of Eq. (11) to laminar flow.

FIGURE 7 PARAGENESIS DIAGRAM OF SLUDGE FLOW



Mr. McPherson's contribution to the discussion is appreciated. Reference the diagram of sheering stress in a pipe on page 13 of reference (1), it may be said that the linear distribution down to zero stress at pipe center is a standard one.[23] It is true as far as relative motion is concerned, the straight lines stop at the yield value. But for a hydrostatic distribution of pressure, the whole stress picture remains the same with zero value at pipe center. Secondly, Mr. McPherson's point of "absolute calibration characteristics" is not quite clear, at least to the writer. In general, experimental data are made comparable and transferrable by observing the principles of dynamic similarity and dimensional homogeity. Otherwise, the experiment is considered incomplete and the data have very limited field of application. Though there are no ideally perfect data at the incomplete stage of development of hydraulic sciences, there is no reason to refuse the comparison of similar values in order to find some common denominators. The main factor in effecting data is the physical condition itself. This may be illustrated by the fact that the yield and the rigidity values of a sludge vary with the pipe diameter which Mr. McPherson accommodated by an arbitrary factor of four. The true phenomenon is that, the total laminar shear in the boundary layer varies with the circumference of the pipe. Now about the critical Reynolds number, that sentence of the writer's quotated by Mr. McPherson was framed on the statement of Messrs. Babbit and Caldwell; "Since sludge possesses no definite viscosity, but a varying apparent viscosity, . . . Reynolds criterion for critical velocity cannot be directly evaluated for sludge." Also sound reason for taking 2,000 and 3,000 as the lower and the upper values were given, and these figures are well within the range of general laboratory practice.[24,25] Pending new discovery, nothing better can be recommended for application.

It may be repeated in brief that in solving any sewage sludge problem, the first important step is to determine the types of flow, plastic or suspension, in order to select the proper set of formulas. For the parameters, the empirical curves on Figs. 1 to 3 are helpful in making proper choices, unless data from direct experiments are on hand. For the turbulent plastic flow, the curve on Fig. 4 may err on the conservative side, and fortunately such flow does not occur very often in practice.

Additional Bibliography

21. Daily, James W. and Bugliurello, George, "The Characteristics of Flow with Dilute Fibre Suspensions, A progress Report", Technical Note, No. 2, Dec. 1957, Hydrodynamics Laboratory, Department of Civil and Sanitary Engineering, Massachusetts Institute of Technology (unpublished).

22. Chou, Tsung-Lien, "Transport of Bed Load in Rivers and Estuaries by Moded Study", Presented to the Faculty of Engineering, Manchester University, Manchester, England, as partial fulfillment of Ph.D. requirements, 1936.

23. Bakmeteff, B. A. "The Mechanics of Turbulent Flow", Princeton University Press, Princeton, N. J., 1936.

24. Schlichting, Herman, "Boundary Layer Theory", Pergamon Press, New York, 1955.

25. Goerther, H., "Boundary Layer Research", Springer Verlag, Berlin, 1958.

SLEUTHING THE BEHAVIOR OF A RIVER[a]

Closure by Edward J. Cleary

EDWARD J. CLEARY,[1] F. ASCE.—Much inspiration in the art of river-quality management is to be derived from experiences on the Thames, "England's Royal River", and on that great European waterway, the Rhine. The author is happy indeed that his paper elicited detailed accounts of current practice on both these streams.

With characteristic modesty, Mr. G. E. Walker, describes the inspection system that has been perfected by the Thames Conservancy on the basis of more than 100 years experience in water-pollution control. Probably nowhere in the world is there a stream which is more conscientiously safeguarded. And no pollution-control agency enjoys greater public respect and confidence than do the Conservators of the Thames.

In addition to the inspection staff on land, which makes frequent and regular contact with each municipality and industry that discharges effluents, the Thames Conservancy maintains patrol cruisers on the river itself. Proudly flying a red burgee, and manned by a uniformed crew, these patrol boats give notice to all that the Conservators are ever alert and will tolerate no deviation from prescribed regulations. Here we have the example of an agency dedicated to a public trust, and competently staffed and directed to perform its duties.

The efforts on the Rhine River for the control of pollution, as described by Dr. Dieterich, are complicated by international relationships. Five nations are endeavoring to integrate their interests and responsibilities. This laudable effort, which resulted in the creation of the International Commission for the Protection of the Rhine River against Pollution, had its origin in 1952.

When we consider the years of discussion and negotiations that has surrounded efforts in the U. S. A. to promote interstate cooperation on pollution control (more than half a century for federal legislation and at least twenty years in the case of one interstate compact), the accomplishments on the Rhine compel admiration.

And not the least of the credit must go to Prof. Otto Jaag, director of the Swiss Federal Institute for Water Supply. Professor Jaag has been chairman of the five-nation Rhine River Commission ever since it was organized. His astuteness as a scientist and his skill as a diplomat led to the establishment of a water-quality monitoring program as one of the first steps in the Rhine program.

Students of the administration of water resources, among which the author counts himself, are indebted to Mr. Walker and Dr. Dieterich for their commentary.

---

a. Proc. Paper 1847, November, 1958, by Edward J. Cleary.

1. Executive Director and Chf. Engr., Ohio River Valley Water Sanitation Commission, Cincinnati, Ohio.

AN ANALOG COMPUTER FOR THE OXYGEN SAG CURVE[a]

Closure by M. D. Sinkoff, C. D. Geilker and J. G. Rennerfelt

M. D. SINKOFF,[1] A. M. ASCE, C. D. GEILKER,[2] and J. G. RENNERFELT.[3] —The authors wish to acknowledge Mr. Babcock's interesting discussion. Pneumatic computers certainly are applicable to many problems and they are being used in installations where mechanical analog, electrical analog, or even digital computers would not be as suitable. It is felt, however, that their application to the area of pollution control would be limited, as would simple analogs of the electrical variety.

The ultimate goal in the adaptation of computers to stream pollution research and control is to simulate a reach of stream, or indeed a river itself, so that confident predictions as well as constant control may become a reality. It is believed that data from automatic dissolved oxygen analyzers and similar devices may some day become input for computer systems thus automating a large portion of pollution control field work.

a. Proc. Paper 1850, November, 1958, by M. D. Sinkoff, C. D. Geilker and J. G. Rennerfelt.

1. A. San. E., Water Pollution Control, San. Engr. Center, Public Health Service, U. S. Dept. of Health, Education and Welfare; now, Applied Science Representative, International Business Machines Corporation, Newark, N. J.
2. Asst. San. Engr., Training Program, San. Engr. Center, Public Health Service, U. S. Dept. of Health, Education and Welfare, Cincinnati, Ohio.
3. Chem. Engr., Skogsindustriernas Vattenlaboratorium, Stockholm, Sweden.

TREATMENT OF LIQUID RADIOACTIVE WASTES[a]

Discussions by R. H. Burns and L. Carlbom

R. H. BURNS.[1]—Since some of the practices as reported at Harwell are inaccurate, the following notes are submitted to correct the confusion that may have resulted:

1. The main bulk of low level liquid waste is treated with caustic soda (to pH 9.5 to 10) and sodium tri-phosphate (50 - 100 mg/1 PO_4^{-3}). After treatment the effluent passes through clarifiers (settlers not sludge blanket precipitators). The clarified effluent is then brought to pH 7 to 7.5 in order to permit discharge. This simple treatment removes about 90 - 95% of the alpha activity and approximately 60 - 85% of that due to β emitters.
2. If the alpha activity is high the effluent is first acidified with sulphuric acid (pH 3.0) followed by 10 mg/1 commercial tannic acid. After mixing the pH is raised to 9.0 with lime and approximately 100 mg/1 PO_4^{-3} added.
3. The effluents from certain buildings which cannot be decontaminated by this simple process, either due to the activity level or on account of other objectionable constituents (high solids content, detergents, etc.) are segregated and treated in a smaller plant. A full description of this was given in Geneva Paper No. 308. A brief summary of the plant and processes used is as follows:

<u>1st Stage</u>	pH raised to 11.5 with caustic soda 50 mg/1 Ca^{+2} 80 mg/1 PO_4^{-3} 40 mg/1 Fe^{2+} The treated effluent is passed through a sludge blanket precipitator.
<u>2nd Stage</u>	The effluent from Stage 1 is treated with— 20 mg/1 Fe^{+2} + 20 mg/1 S^{-2} and passed through a second sludge blanket unit.
<u>3rd Stage</u>	The effluent from Stage 2 is passed through vermiculite beds.

The overall removal varies slightly depending on the anionic or non-ionic activity present, but the average removals over the last six months have been:

---

a. Proc. Paper 1930, January, 1959, by Conrad P. Straub.

1. Industrial Chemistry Group, Atomic Energy Research Establishment, United Kingdom Atomic Energy Authority, Harwell, England.

α emitters, 99.98%
β emitters, 99.6 %

4. The dewatered sludges are taken to the South coast and dumped in a recognized disposal area where the depth of water averages 100 fathoms.
5. Small volumes of high activity level are now absorbed onto exfoliated vermiculite before being placed in shielded sea disposal containers.
6. The process ascribed to Dr. Amphlett was worked out mainly by a member of my Group. Some of the ideas are being used including the electrolytic de-ionisation cell. This is, however, a very small unit (50 g.p.h.) and much work still remains to be done on this process. I am at the present time installing a H.F. furnace one of the uses for which will be to study the practicability of firing the chemical sludges and other wastes in a non-leachable solid mass.
7. Our experience with vermiculite is that, whereas laboratory experiments indicated that breakthrough of cesium would occur after about 250 bed volumes, on the plant 50 bed volumes were passed before any significant increase in activity level of the effluent took place. The effluent was, in fact, still acceptable after 1300 bed volumes.

L. CARLBOM.[1]—Additional comments to supplement the material presented follow:

The quantities reported in Table I were estimates referring to a hypothetical plutonium separation plant. These figures have now been revised and nothing could be said today about the volumes and activity levels of wastes in Categories I and II.

Category I wastes will be stored in acid proof tanks in underground excavations. Category II wastes will probably be concentrated and stored in tanks for some years before the release to the waste treatment plant (Categories III and IV). Category III will be brought to an evaporator or combined with Category IV depending on the activity level. Category IV is regarded as normally active and will be treated by flocculation. Categories V, VI, and VII are ordinarily non-active but if activity occurs these Categories will be brought to the treatment plant as Category IV. Category VIII is supposed never to be active and is not normally controlled. The treatment plant handles in fact two active categories: a. Medium active effluent (Category II + III); and b. Low active effluent (Category IV).

1. Reactor Development Division, Aktiebolaget Atomenergi, Stockholm, Sweden.

THE HYDROLOGY OF URBAN RUNOFF[a]

Discussions by M. B. McPherson and M. Joseph Willis, Paul Bock, and Carl F. Izzard and Charles Armentrout

M. B. McPHERSON,[1] M. ASCE and M. JOSEPH WILLIS,[2] M. ASCE.—Some of the latest theory has been ingeniously incorporated in the authors' analyses in a clear and elegant manner, taking full advantage of modern calculating equipment. Nearly all of the assumptions and limitations characterizing the several elements of the analyses have been well defined. It must be noted that the numerical results presented by the authors represent combinations of synthetic and hypothetical arguments applied to specific complexes of urban development without benefit of a broad experimental base or verification by means of field measurement. Despite the large number of hypothetical conditions and assumptions, which cannot be avoided in this type of analysis, the assemblage of the several anatomical constituents into a hydrological whole gives the reader a better comprehension and appreciation of the separate factors which contribute to an urban runoff hydrograph.

The many procedural steps of the paper can be consolidated into four principal parts for the purpose of discussion: (1) synthetic hyetograph; (2) conveyance of rainfall to outlet of laterals; (3) routing of flows in main sewers; (4) attenuation of rainfall with distance from the storm center. One of the writers has commented on the synthetic hyetograph in an earlier discussion.(1) In connection with the second item it should be noted that the overland flow calculations were based upon experimental tests on sprinkled plats of homogeneous cover with uniform rates of rainfall supply. As a consequence any application of these laboratory tests to a continuously varying rainfall supply on a heterogeneous cover must be regarded as a possible gross over-extension beyond the limits of the original experiments. According to the techniques employed, the routing of flows in the main sewers appears to be the part subjected to the least number of assumptions. The section on "Areal Distribution of Rainfall" and the accompanying figure (Fig. 24) do not include sufficient documentation and explanation.

The Johns Hopkins University has conducted field measurements over the past ten years of urban storm water runoff in Baltimore, Maryland.(2) Peak flows at a large number of inlets and in several main sewers draining areas up to 153 acres in size have been recorded simultaneously with concomitant storm rainfall. This project represents the most extensive measurement program attempted in this country. Unfortunately a direct comparison of the field measurements with the authors' calculations for smaller areas cannot be made since all gaged area roofs drain directly into the gutters as opposed to

---

a. Proc. Paper 1984, March, 1959, by A. L. Tholin and Clint J. Keifer.
1. Research Engr., Philadelphia Water Dept., Philadelphia, Pa.
2. Assoc. Prof. of Civ. Eng., Swarthmore College, Swarthmore, Pa.

the direct connections to sewers in Chicago. The results of the Johns Hopkins study have indicated a strong relation between peak runoff and (1) the percentage of total imperviousness of the drainage area, (2) the maximum five-minute intensity of the storm, and (3) the duration of the intense part of the storm. There is unfortunately no direct mathematical correlation between the last two items. Although the "directly connected" imperviousness of Type 5 Land Use is 36% (Fig. 9), the total imperviousness appears to be 46.3%. The writers have applied their own interpretation and abbreviation of the Johns Hopkins method of analysis to the case of 46.3% total imperviousness, and have found that the peak flow at "outlet of lateral" would be about 160% of the 1.8 cfs/acre in Fig. 20(a). The following values were utilized: (a) maximum five-minute average intensity of 5.9 in./hr from the upper curve of Fig. 3; (b) a duration of the intense part of the storm of 20 minutes (determined by the writers to be reasonably consistent with the theoretical 5-year recurrence interval sewer discharge rate for both Baltimore and Philadelphia; referring to the mass curve of rainfall in Fig. 5(a), 20 minutes appears to be a reasonable estimate for the synthetic hyetograph); (c) a simplified generalization of the time-distance attentuation procedure employed at Johns Hopkins; and (d) a "lateral outlet" drainage area of 10 acres, with a time of travel in the lateral of seven and one-third minutes. Other combinations were used to establish a range of estimated peak flow values, but the results were not significantly different. It would be improper to conclude that the divergence in answers is due solely to different types of roof connections. There is presently no real basis for comparison. If the authors could rerun the calculations for Type 5 Land Use under Fig. 20(a) restrictions for the case of roof drains connected to gutters, a more sound basis for comparison with field measurements might then be available. (Fig. 20(a) and Type 5 are suggested since there are more details on these combinations given in the paper). This request is an obvious imposition, but the great potential value of this information should be adequate justification. Both an inlet and a lateral outflow hydrograph would be quite useful, together with the inlet hydrograph not shown in Fig. 18 (it appears that column 5, Table IV is for the Type 5 inlet hydrograph). It would also be interesting to know the total imperviousness of the other three land use types.

The "rational" method implies that the attenuation of peak flow in a sewer system is related directly to the declination of an intensity-duration curve. Since intensity-duration curves represent a statistical manipulation of individual intensities for each duration arbitrarily separated from the sum total of selected point rainfall data, there is no logical way to 'rationalize' any basic principles for the "rational" method. The designer who has been obliged to use the completely empirical "rational" method for lack of anything better should derive some comfort from Fig. 19. The routing calculations show that there is a similar trend in the time-distance attenuation of the curves for the two different methods. However, the rates of change of the curves and the absolute ordinate values offered by the authors must be regarded as pure estimates pending generalized substantiation via carefully controlled field measurements.

Turning to Figs. 20 and 21, it is apparent that each curve is a trace through the four peak runoff values determined from the calculations for each of the four land use types. The authors thereafter appear to have extrapolated the curves to an imperviousness of 100%. What bases or additional assumptions were used for this purpose?

Perhaps the most influential factor from a cost/design standpoint is the "areal distribution of rainfall" given in Fig. 24. The authors have failed to explicitly define the ambiguous term: "Duration." Since a large share of the pertinence and value of their paper hinges about an acceptance of Fig. 24, and since the source of information is "an unpublished report" not available to the public, it is hoped that the authors will enlarge considerably on their coverage of this subject in their closure to the discussions of their paper. It is evident that Fig. 25 was obtained through an interpretation of Fig. 24, but the methodology used has been implied and not demonstrated. Under the section "Sample Computation Method for Large Drainage Areas" they cite abruptly a "640 acre limit for use of uniform areal distribution of rainfall," a limit which is not mentioned or explained in any prior section. From this and a succeeding remark it appears that the curves on the right side of Fig. 25 are not considered applicable to areas less than one square mile, or were possibly so treated. Inasmuch as neither Fig. 24 nor Fig. 25 are mentioned in connection with the developments leading to Figs. 20 and 21, the question arises as to exactly what allowance, if any, was provided for areal distribution in the calculations leading to Figs. 18, 19, 20 and 21. If no allowance for areal distribution was made for the analysis curves in Fig. 19, has a fair comparison with the "rational" method been given in this figure?

Over a year ago the authors kindly supplied the first writer with data on the times of occurrence of peak flows from their calculation files for all four land use types, to supplement the data of Figs. 17 and 18. These data have been plotted in Fig. TK, and represent the calculation conditions applied to Fig. 20(a). The time-displacement of peaks is seen to be approximately 0.7 of the time of travel. A value of 2/3 was used by Hicks(3) in an earlier hydrograph analysis. In the analytical procedure employed by Johns Hopkins personnel(2) a value of 0.8 is used. A recent review of the Johns Hopkins field data files by Mr. Viessman has indicated that the peak flow in Baltimore inlets has generally occurred about one minute after the most intense burst of rainfall; the arbitrary extrapolation of data in Fig. TK for the three most impervious land use types indicates a peak flow time at the inlet 1-1/2 minutes after the peak intensity of the synthetic hyetograph. (In Table IV, the peak lateral inflow occurs at 68.5 min.) If the authors recalculate the Type 5 Land Use values for the situation with all house roof drains connected to gutters, it is hoped also that a comparison with the characteristics of Fig. TK will be made using those results, to further extend the value of the recalculations.

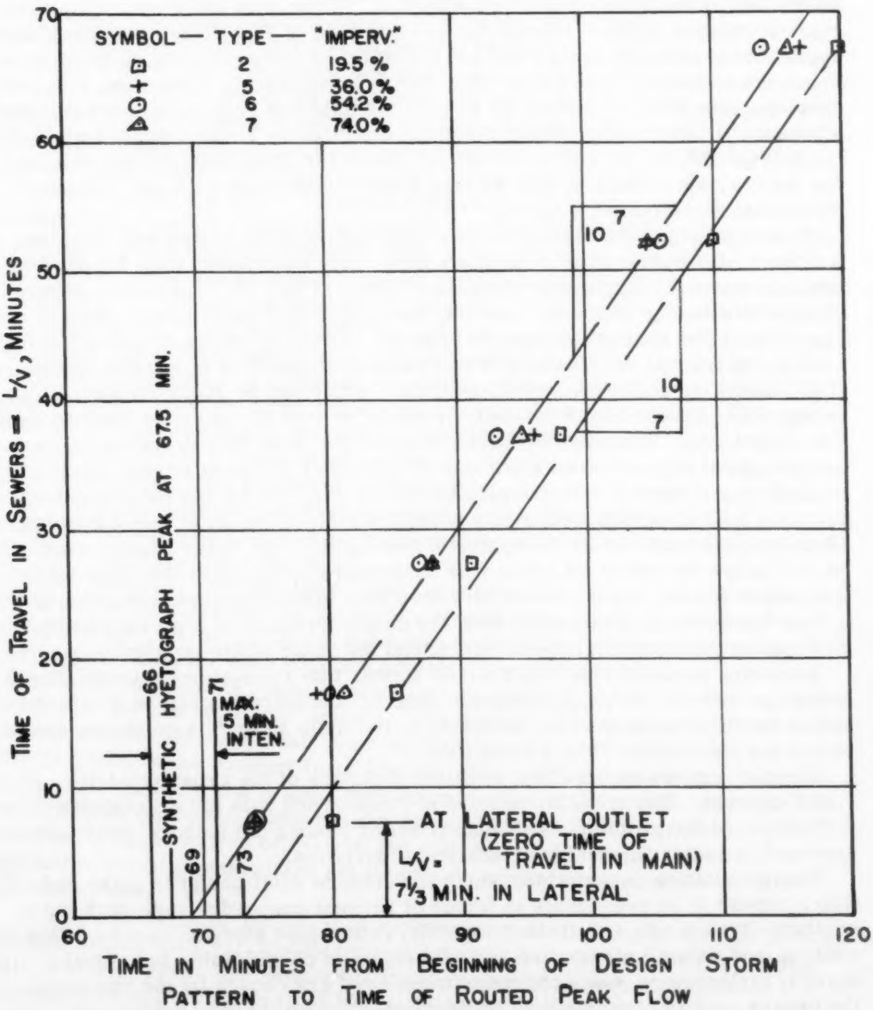
Referring again to Fig. TK, it would appear that the statement under Step 8, second paragraph: "The approximate time of concentration can be obtained by adding ten minutes to the time of travel in the main sewer" is quite reasonable except for the case of Type 2 Land Use.

The paper gives an excellent qualitative picture of the urban rainfall-runoff process. However, the writers are concerned with the quantitative difficulties of the problem which may possibly render the authors' qualitative approach unusable for design or analytical purposes.

The quantitative assumptions which the authors must perforce make and which appear to be reasonable in terms of present knowledge regarding the synthetic hyetograph, infiltration capacity, depression storage, overland flow, routing, and rainfall attenuation are all subject to considerable individual errors; furthermore, since the magnitudes used are chosen by the engineer, the errors may on occasion tend to be cumulative rather than having any compensating tendencies. When it is considered that the representative

FIGURE TK
(DISC. OF 1984)

TIME—ATTENUATION FROM ROUTINGS FOR CONDITIONS OF FIG. (20a)



infiltration curve chosen by the authors may under certain conditions be of the order of magnitude of 50% or more in error, and bearing in mind the possibility of errors of similar magnitude in the manipulation of data for the hyetograph, depression storage, overland flow, and rainfall attenuation, it is quite conceivable that a final computed value of runoff might be more unrealistic than the runoff calculated by the "rational" method through the selection of a single all-embracing coefficient. The writers believe that the value of the paper would be enhanced by a discussion of the magnitude of the errors likely to occur, at the present state of the art, in each step of the authors' method.

The writers think that the authors have succeeded in their avowed aim: to "guide the thinking of the sewer designer to a clearer visualization of what happens, hydrologically, when rain falls on the heterogeneous catchment surfaces of a city;" however, the extensive calculations contained in the paper may lead many readers to expect that the approach used by the authors for qualitative clarification of the rainfall-runoff process may soon come of age quantitatively. The writers question whether this is possible for some time to come, considering how much remains to be learned about the mechanics of infiltration, detention, overland flow, etc. It is suggested that a simpler approach based on actual measurements of rainfall-runoff relationships from urban drainage areas might prove much more fruitful until such time as sufficient knowledge is gained of all components of the urban runoff process. By then it may be possible to develop a computing procedure approaching the qualitative model. On the other hand, because of the many factors inherent in evaluation using the quantitative model, it would seem that a simpler method of calculation backed by extensive field measurements holds more promise for the future. The present field program of the City of Philadelphia includes seven urban gaging stations for drainage areas from 8 to 260 acres in size.

(Note: Reference 6 was published in 1946, not 1956.)

REFERENCES

1. Discussion of "Synthetic Storm Pattern for Drainage Design, by C. J. Keifer and H. H. Chu, ASCE Proc. Paper No. 1332, Aug. 1957," by M. B. McPherson, ASCE Proc. Paper No. 1558, pp. 49-57, Feb. 1958.

2. "Progress Report on the Storm Drainage Research Project, July 1, 1956 to June 30, 1958" by Paul Bock and Warren Viessman, Jr., Dept. of Sanitary Engineering and Water Resources, The Johns Hopkins University, Baltimore, Md., July 1958.

3. "A Method of Computing Urban Runoff," by W. I. Hicks, Trans. ASCE No. 109, Paper No. 2230, pp. 1244-1245, 1944.

PAUL BOCK.[1]—The Chicago Hydrograph Method is an example of the "Microscopic Approach" to urban storm drainage design. In general, the Microscopic Approach attempts to define a chronological design storm pattern and then quantify all pertinent physical phenomena from the input (rainfall) to output (runoff). To evaluate the individual phenomenon the drainage area is

1. Associate Prof., Dept. of Civ. Eng., University of Delaware; Newark, Del.

Table I

Time Distribution of Greatest 15 Minute Duration Excessive Precipitation Rates (Above 2 year Frequency) at Baltimore, Maryland (1894-1935 inclusive)

Order of Mag.	Date		Time From Start of Excessive Precipitation Rate in Minutes 5	10	15
1[2/]	7/12/03	% Ins.	0.34	0.73	1.00
		Ins.	0.65	1.39	1.90
2[6/]	8/25-26/11	% Ins.	0.41	0.63	1.00
		Ins.	0.65	0.99	1.57
3[5/]	7/18/07	% Ins.	0.32	0.67	1.00
		Ins.	0.45	0.99	1.43
4	7/7/25	% Ins.	0.37	0.78	1.00
		Ins.	0.46	0.97	1.24
5	7/25/01	% Ins.	0.26	0.68	1.00
		Ins.	0.31	0.80	1.18
6[7/]	5/24/30	% Ins.	0.20	0.63	1.00
		Ins.	0.22	0.71	1.12
7	7/17/97	% Ins.	0.25	0.63	1.00
		Ins.	0.26	0.66	1.05
8	8/6/19	% Ins.	0.15	0.56	1.00
		Ins.	0.16	0.58	1.04
9	7/7/05	% Ins.	0.29	0.77	1.00
		Ins.	0.30	0.79	1.03
10[1/]	8/27/02	% Ins.	0.36	0.77	1.00
		Ins.	0.37	0.79	1.02
11	8/15/34	% Ins.	0.34	0.64	1.00
		Ins.	0.35	0.64	1.02
12	8/12/34	% Ins.	0.43	0.89	1.00
		Ins.	0.43	0.89	1.01
13	9/1/10	% Ins.	0.36	0.73	1.00
		Ins.	0.35	0.72	0.98
14	8/31/18	% Ins.	0.55	0.76	1.00
		Ins.	0.52	0.72	0.95
15	6/13/15	% Ins.	0.30	0.63	1.00
		Ins.	0.28	0.59	0.94

divided into different types of surfaces; then the flow from each surface is computed and the individual flows synthesized by routing techniques to obtain the outflow hydrograph. The main advantages of the authors' method are: its generally sound physical approach, its susceptibility to systematic improvement as knowledge increases, and its "clearer" visualization of what happens, hydrologically, when rain falls on the heterogeneous catchment surfaces of a city." Its principal disadvantage for practical engineering applications is that it requires the use of numerous assumptions, simplifications, untested individual coefficients, and other data not generally available.

On the other hand, the Rational Method is an example of the "Macroscopic Approach" which considers a drainage area as being homogeneous through use

Table I (cont.)

Time Distribution of Greatest 15 Minute Duration Excessive Precipitation Rates (Above 2 year Frequency) at Baltimore, Maryland (1894-1935 inclusive)

Order of Mag.	Date		Time From Start of Excessive Precipitation Rate in Minutes 5	10	15
16	8/5/08	% Ins.	0.33	0.77	1.00
		Ins.	0.31	0.72	0.94
17	8/25/05	% Ins.	0.31	0.68	1.00
		Ins.	0.28	0.62	0.91
18	9/19/96	% Ins.	0.33	0.72	1.00
		Ins.	0.30	0.65	0.90
19 3/	7/30/03	% Ins.	0.17	0.68	1.00
		Ins.	0.15	0.61	0.90
20 8/	6/27/34	% Ins.	0.22	0.51	1.00
		Ins.	0.20	0.46	0.90
21	9/23/31	% Ins.	0.28	0.60	1.00
		Ins.	0.25	0.53	0.88

Note	Time after start of excessive rate	Amt. of rainfall prior to maximum 15 minute rate during excessive rate
1/	5 minutes	0.10
2/	5 minutes	0.33
3/	5 minutes	0.03
4/	15 minutes	0.20
5/	10 minutes	0.60
6/	10 minutes	0.75
7/	5 minutes	0.18
8/	5 minutes	0.13

of an overall weighted runoff coefficient, flow from a drainage area considered only at its outflow point, and rainfall assumed to be acting uniformly in time and space. The essence of the Rational Method is the judicious selection of the runoff coefficient which "must come out of years of experience in a particular locality and cannot be adequately nor easily conveyed to a novice." The advantages of the Rational Method are its simplicity of application and, apparently, popular faith in its efficacy. Its disadvantages are that its concepts are physically unsound, ambiguous, and untested (for example, the writer is unaware of any measurements of "time of concentration" on urban areas as defined by the authors); it is inherently not susceptible to improvement; and its predictions of peak flows have been found to be unreliable as based upon measurements of rainfall-runoff.(1,2,3,4,5)

Both the authors' method and the Rational Method assume a "design storm" as a first step. Although not generally stated, the design storm of the Rational Method is assumed to be of uniform intensity "i" lasting for the time of concentration. Because "i" is computed as the total amount of rainfall (in inches) divided by the duration (in hours), it represents an average rainfall rate which is constant throughout the duration. Again a uniform rainfall rate is implied because of the preclusion of values of runoff coefficient greater than 1. Thus if the Rational design storm were considered non-uniform, then

Table II

Time Distribution of Greatest 30 Minute Duration Excessive Precipitation Rates (Above 2 year Frequency) at Baltimore, Maryland (1894-1935 inclusive)

Order of Mag.	Date		Time From Start of Excessive Precipitation Rate in Minutes 5	10	15	20	25	30
1	7/12/03	Ins.	0.33	0.98	1.72	2.23	2.52	2.69
		% Ins.	0.12	0.36	0.64	0.83	0.94	1.00
2	8/25-26/11	Ins.	0.19	0.75	1.30	1.74	2.22	2.30
		% Ins.	0.08	0.33	0.57	0.76	0.97	1.00
3	7/18/07	Ins.	0.30	0.60	1.05	1.59	2.03	2.20
		% Ins.	0.13	0.27	0.48	0.72	0.92	1.00
4	7/25/01	Ins.	0.31	0.80	1.18	1.54	1.77	1.85
		% Ins.	0.16	0.43	0.64	0.83	0.96	1.00
5 4/	7/12/11	Ins.	0.16	0.29	0.46	0.71	1.07	1.31
		% Ins.	0.12	0.22	0.35	0.54	0.82	1.00
6	5/24/30	Ins.	0.18	0.40	0.89	1.30	1.43	1.43
		% Ins.	0.13	0.28	0.62	0.91	1.00	1.00
7	9/1/10	Ins.	0.35	0.72	0.98	1.11	1.28	1.39
		% Ins.	0.25	0.52	0.71	0.80	0.92	1.00
8	8/1/13	Ins.	0.15	0.36	0.67	1.01	1.24	1.38
		% Ins.	0.11	0.26	0.48	0.73	0.90	1.00
9 6/	7/2/22	Ins.	0.22	0.59	0.78	0.89	1.14	1.38
		% Ins.	0.16	0.43	0.57	0.65	0.83	1.00
10 1/	8/26/89	Ins.	0.21	0.45	0.56	0.78	0.96	1.35
		% Ins.	0.16	0.33	0.41	0.58	0.71	1.00
11	9/15/00	Ins.	0.18	0.60	0.86	1.02	1.18	1.32
		% Ins.	0.14	0.45	0.65	0.77	0.90	1.00
12	8/6/19	Ins.	0.16	0.58	1.04	1.20	1.27	1.30
		% Ins.	0.12	0.45	0.80	0.93	0.98	1.00
13 3/	7/7/05	Ins.	0.07	0.37	0.86	1.10	1.19	1.28
		% Ins.	0.05	0.29	0.67	0.86	0.93	1.00
14 5/	7/10/17	Ins.	0.13	0.30	0.66	0.94	1.16	1.28
		% Ins.	0.10	0.23	0.51	0.73	0.90	1.00

rates greater than "the" average "i" must be present. These higher intensities could cause a runoff rate greater than that computed by using a runoff coefficient equal to or less than unity. Emil Kuichling, originator of the 1889 Rational Method, assumed that the rainfall rates were steady. Without benefit of recording rain gages he concluded he could "form tolerably accurate estimates of the rate of precipitation from both the sound of the rain upon the roof of the building and the appearance of the street gutters."(6) Modern rain gagings indicate that rainfall seldom, if ever, is steady.

The design storm used in the Chicago Hydrograph Method is a chronological storm pattern derived from two sources: (a) eighty-three station rainfalls (considered excessive) for durations of 15, 30, 60, and 120 minutes for obtaining the relative time position "r" of the peak rainfall rate, and (b) the local rainfall intensity-duration-frequency curve for obtaining the 5 year intensities. Although the actual hyetalgraphs are not shown, the data tabulated in Table 1(7) indicate the ordinal position of the peak five minute rainfall for 15, 30, 60, and 120 minute durations. For example, the ordinal positions for the 120 minute

Table II (cont.)

Time Distribution of Greatest 30 Minute Duration Excessive Precipitation Rates (Above 2 year Frequency) at Baltimore, Maryland (1894-1935 inclusive)

Order of Mag.	Date		Time From Start of Excessive Precipitation Rate in Minutes 5	10	15	20	25	30
15	6/13/15	Ins.	0.28	0.59	0.94	1.06	1.11	1.25
		% Ins.	0.22	0.47	0.75	0.85	0.89	1.00
16 7/	10/10/22	Ins.	0.27	0.43	0.65	0.93	1.09	1.25
		% Ins.	0.21	0.34	0.52	0.74	0.87	1.00
17	8/27/02	Ins.	0.10	0.47	0.89	1.12	1.22	1.24
		% Ins.	0.08	0.38	0.72	0.90	0.97	1.00
18 2/	5/24/03	Ins.	0.37	0.59	0.66	0.72	1.01	1.23
		% Ins.	0.30	0.48	0.54	0.58	0.82	1.00
19	6/27/34	Ins.	0.13	0.33	0.59	1.03	1.20	1.21
		% Ins.	0.11	0.27	0.49	0.85	1.00	1.00
20	8/25/05	Ins.	0.28	0.62	0.91	1.04	1.14	1.20
		% Ins.	0.23	0.52	0.76	0.87	0.95	1.00
21	9/22-23/07	Ins.	0.13	0.33	0.69	0.97	1.13	1.20
		% Ins.	0.11	0.27	0.58	0.81	0.94	1.00

Note	Time after start of excessive rate	Amt. of rainfall prior to maximum 30 Minute rate during excessive rate (inches)
1/	5 minutes	0.04"
2/	5 minutes	0.21"
3/	10 minutes	0.13"
4/	15 minutes	0.46"
5/	5 minutes	0.06"
6/	15 minutes	0.46"
7/	30 minutes	0.56"

duration are: 20, 19, 6, 6, 16, 9, 6, 3, 15, 6, 12, 5, 13, 21, 3, 3, 15, 10, 10, 15, 4, 10, 18, 4, 2, 2, 6, 9, 7, 4, 24, 5, 4, 15, 3, 5. The mean ordinal position is 9.30 which yields a mean location of the peak within rainfall duration, "r" = 0.366. The large value of the standard deviation of the ordinal positions, $\sigma = 6.1$, indicates that the design value, r = 3/8 is highly questionable. Since the Chicago design storm is based on a 180 minute duration, it appears appropriatè that "r" should also be computed for this duration.

The authors have quite correctly not attached a frequency to the Chicago "Design Storm Pattern" although Fig. 3 might suggest that it is a "hyetalgraph of '5 year' rainfall" with an average frequency of occurrence of once in 5 years. The chronological pattern itself would have a frequency considerably rarer than 5 years, although the intensities for durations from 5 to 180 minutes correspond to a 5 year frequency. Since the Design Storm Pattern has no assigned frequency (in the usual statistical sense), it would be revealing for the authors to discuss their "engineering reasons" for selecting the 5 year rainfall-intensity-duration frequency curve shown in Fig. 3.

In a previous study,(8) the writer examined the time distribution of excessive precipitation rates for Baltimore from 1894 to 1935. During this period, the U. S. Weather Bureau data were published as "accumulated depths for consecutive periods of time." This convenient tabulation makes it possible to determine whether the rainfall rates were uniform or variable over durations of excessive rates. The 21 storms containing the greatest rainfall

Table III

Time Distribution of Greatest 60 Minute Excessive Rainfall Rates (Above 2 year Frequency) at Baltimore, Maryland (1894-1935 inclusive)

Order of Mag.	Date		Time From Start of Excessive Precipitation Rate in Minutes 5	10	15	20	25	30	35	40	45	50	60
1	7/12/03	Ins.	0.33	0.98	1.72	2.23	2.52	2.69	2.87	2.87	2.87	2.87	2.87
		% Ins.	0.11	0.34	0.60	0.78	0.88	0.94	1.00	1.00	1.00	1.00	1.00
2	7/18/07	Ins.	0.30	0.60	1.05	1.59	2.03	2.20	2.38	2.61	2.72	2.72	2.72
		% Ins.	0.11	0.22	0.39	0.59	0.75	0.81	0.88	0.96	1.00	1.00	1.00
3	8/25-26/11	Ins.	0.19	0.75	1.30	1.74	2.22	2.30	2.30	2.30	2.30	2.30	2.30
		% Ins.	0.08	0.33	0.57	0.76	0.97	1.00	1.00	1.00	1.00	1.00	1.00
4[1/]	10/4/22	Ins.	0.22	0.50	0.66	0.82	0.92	1.03	1.13	1.24	1.44	1.63	2.02
		% Ins.	0.11	0.25	0.33	0.41	0.46	0.51	0.56	0.62	0.72	0.81	1.00
5	7/12/11	Ins.	0.11	0.30	0.46	0.62	0.75	0.92	1.17	1.53	1.77	1.88	1.95
		% Ins.	0.06	0.15	0.24	0.32	0.38	0.47	0.60	0.78	0.91	0.97	1.00
6	7/25/01	Ins.	0.31	0.80	1.18	1.54	1.77	1.85	1.91	1.94	1.94	1.94	1.94
		% Ins.	0.16	0.41	0.61	0.80	0.91	0.95	0.98	1.00	1.00	1.00	1.00
7	7/2/22	Ins.	0.22	0.34	0.46	0.68	1.05	1.24	1.35	1.60	1.84	1.84	1.84
		% Ins.	0.12	0.19	0.25	0.37	0.57	0.67	0.73	0.87	1.00	1.00	1.00
8	8/3-4/15	Ins.	0.10	0.29	0.48	0.60	0.76	0.92	1.15	1.37	1.45	1.47	1.69
		% Ins.	0.06	0.17	0.28	0.35	0.45	0.54	0.68	0.81	0.86	0.87	1.00
9	9/15/00	Ins.	0.18	0.60	0.86	1.02	1.18	1.32	1.46	1.49	1.55	1.58	1.61
		% Ins.	0.11	0.37	0.54	0.64	0.73	0.82	0.91	0.92	0.96	0.98	1.00
10	8/26/99	Ins.	0.04	0.25	0.49	0.60	0.82	1.00	1.39	1.51	1.56	1.56	1.56
		% Ins.	0.02	0.16	0.31	0.39	0.53	0.64	0.89	0.97	1.00	1.00	1.00
11	5/24/03	Ins.	0.21	0.58	0.80	0.87	0.93	1.22	1.44	1.51	1.51	1.54	1.54
		% Ins.	0.14	0.38	0.52	0.56	0.60	0.79	0.94	0.98	0.98	1.00	1.00
12	9/14-15/04	Ins.	0.14	0.31	0.54	0.81	1.01	1.14	1.28	1.48	1.53	1.53	1.53
		% Ins.	0.09	0.20	0.35	0.53	0.66	0.75	0.84	0.97	1.00	1.00	1.00
13	9/1/10	Ins.	0.35	0.72	0.98	1.11	1.28	1.39	1.51	1.53	1.53	1.53	1.53
		% Ins.	0.23	0.47	0.64	0.73	0.84	0.91	0.99	1.00	1.00	1.00	1.00
14	8/4/11	Ins.	0.18	0.47	0.59	0.63	0.68	0.76	0.93	1.06	1.18	1.28	1.51
		% Ins.	0.12	0.31	0.39	0.42	0.45	0.50	0.62	0.70	0.78	0.85	1.00

depths for 15, 30, and 60 minute durations in Baltimore for the 42 year period from 1894 - 1935 are listed in Tables I, II, and III respectively, and are plotted in dimensionless form in Figs. 1, 2, and 3 respectively. The scatter appears to be so wide that no meaningful typical chronological pattern can be detected for these storms of design intensity.

The authors have explicitly noted assumptions and simplifications made throughout the development of their method. Many of these, of course, are hidden in the Rational Method's runoff coefficient. It is revealing to list some of the authors' assumptions:

pp. 50, 51	Sidewalks and garage roofs considered as pervious areas.
p. 51	Roofs considered as uniform strips of equivalent area.
p. 53	Evaluation of Design Storm's chronological pattern and magnitudes (implicit assumption).

Table III (cont.)

Time Distribution of Greatest 60 Minute Excessive Rainfall Rates (Above 2 year Frequency) at Baltimore, Maryland (1894-1935 inclusive)

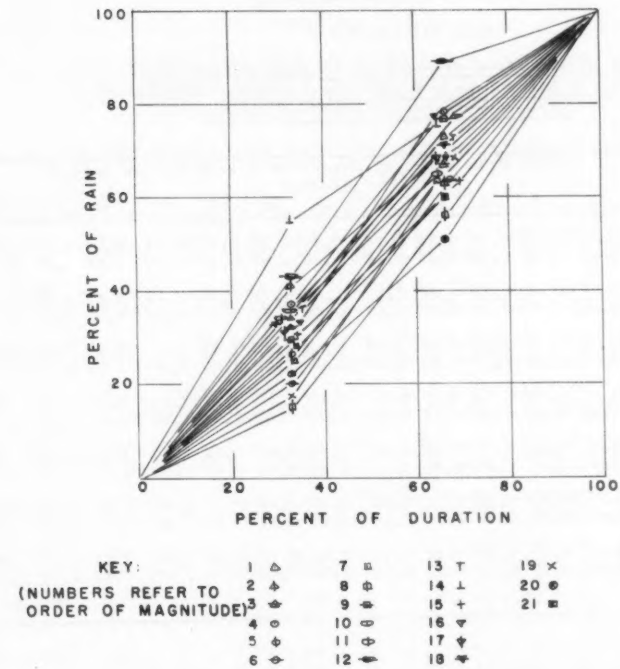
Order of Mag.	Date		Time From Start of Excessive Precipitation Rate in Minutes 5	10	15	20	25	30	35	40	45	50	60
15[2/]	7/22-23/06	Ins.	0.01	0.06	0.13	0.27	0.41	0.55	0.70	0.83	0.97	1.10	1.48
		% Ins.	0.01	0.04	0.09	0.18	0.28	0.37	0.47	0.56	0.66	0.75	1.00
16	7/10/17	Ins.	0.06	0.19	0.36	0.72	1.00	1.22	1.34	1.42	1.46	1.46	1.46
		% Ins.	0.04	0.13	0.25	0.49	0.68	0.83	0.92	0.98	1.00	1.00	1.00
17	5/24/30	Ins.	0.18	0.40	0.89	1.30	1.43	1.43	1.43	1.43	1.43	1.43	1.43
		% Ins.	0.13	0.28	0.62	0.91	1.00	1.00	1.00	1.00	1.00	1.00	1.00
18	7/7/05	Ins.	0.07	0.13	0.20	0.50	0.99	1.23	1.32	1.41	1.41	1.41	1.41
		% Ins.	0.05	0.09	0.14	0.35	0.70	0.87	0.94	1.00	1.00	1.00	1.00
19	8/5-6/02	Ins.	0.10	0.30	0.49	0.56	0.64	0.89	1.24	1.40	1.40	1.40	1.40
		% Ins.	0.07	0.21	0.35	0.40	0.46	0.64	0.88	1.00	1.00	1.00	1.00
20	9/7/34	Ins.	0.06	0.11	0.16	0.25	0.38	0.59	0.90	1.19	1.31	1.36	1.39
		% Ins.	0.04	0.08	0.12	0.18	0.27	0.42	0.65	0.85	0.94	0.98	1.00
21	5/6/35	Ins.	0.10	0.20	0.30	0.43	0.69	0.90	1.04	1.20	1.22	1.23	1.38
		% Ins.	0.07	0.14	0.22	0.31	0.50	0.65	0.75	0.87	0.89	0.89	1.00

Note

1/ Maximum 60 minute excessive rate began 40 minutes after start of excessive rate. Accumulated depth during excessive rate prior to maximum 60 minute excessiver rate = 0.99".

2/ Maximum 60 minute excessive rate began 50 minutes after start of excessive rate. Accumulated depth during excessive rate prior to maximum 60 minute excessiver rate = 1.08".

p. 55	Selection of infiltration capacity curve (implicit assumption).
p. 57	Time of commencement of overland flow and depression storage supply.
p. 61	Overall volume of depression storage.
p. 61	Ogee distribution curve of depression storage.
p. 65	Eq. (11) holds for unsteady flow conditions.
p. 69	Distributions of inflow hydrograph to gutter.
p. 74	Eq. (19) obtained by integration is not mathematically rigorous,(9,10) since choice of differential element of integration affects the computed flow. The conventional form of the equation Q = AV is more valid and gives a 19% lower value of Q for a given value of "n". Published values of "n" have been derived from the equation Q = AV.
p. 74	Steady flow conditions in gutter.
p. 75	Eq. (23) equilibrium conditions for flow in gutter.
p. 77	Uniform velocity distribution in lateral sewer.



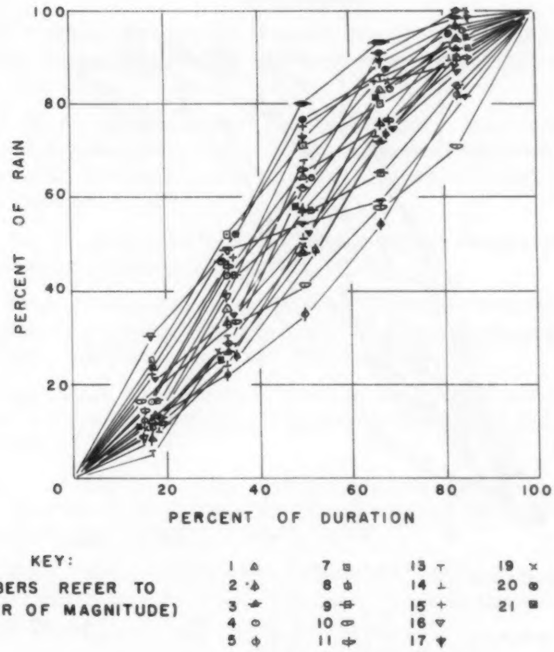
TIME DISTRIBUTION OF GREATEST 15 MINUTE DURATION EXCESSIVE PRECIPITATION RATES (ABOVE 2 YEAR FREQUENCY) AT BALTIMORE MARYLAND (1894-1935 INCLUSIVE)

FIGURE 1

p. 81	Spatial distribution of gutter and roof hydrographs to form total inflow hydrograph.
p. 84	Spatial distribution of lateral hydrograph into main sewer.
p. 89	Time of concentration.

The final answer desired, peak runoff rate, is affected by the accuracy of the determination of all losses and hydraulic phenomena, and the validity of the simplifying assumptions. If the errors are small and non-cumulative, the prediction of the runoff is valid. Otherwise, this detailed approach may give no better "practical" answers than use of a single gross runoff coefficient that "cannot be methodically conceived by any form of mental exercise."

The improvement and verification of any method for urban storm drainage design rests ultimately on measurements of rainfall-runoff on urban areas. At present, the Chicago Hydrograph Method suffers from lack of experimental verification. Ideally, to check this method, there should be sufficient independent data of rainfall-runoff on actual urban areas and elemental units of urban areas to check each of the authors' steps, particularly, steps 3 to 7 inclusive. For example, there should be measurements of infiltration, volume



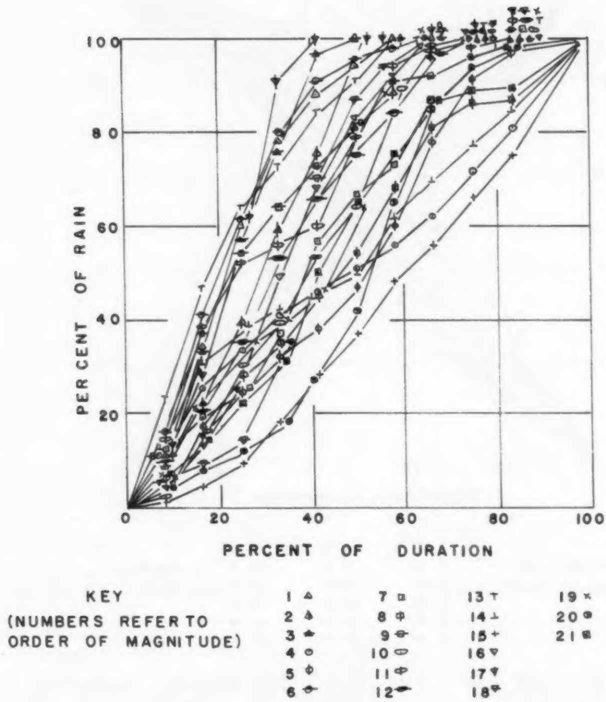
TIME DISTRIBUTION OF GREATEST 30 MINUTE DURATION EXCESSIVE PRECIPITATION RATES (ABOVE 2 YEAR FREQUENCY) AT BALTIMORE MARYLAND (1894—1935 INCLUSIVE)

FIGURE 2

of depression storage, and flows in the gutter, lateral sewers and main sewer outlet.

Unfortunately, there exists a paucity of gagings of rainfall-runoff on urban areas. The dates of the authors' references indicate the serious inadequacy of research on urban drainage. Except for (1), an analytical study of Chicago storm patterns, the specific references are from 11 to 19 years old. Only one applies particularly to urban drainage and none is adequately supported by actual field gagings on urban areas. Where urban gagings have been made, the gagers (almost without exception) have introduced methods of their own based upon the experimental data. Likewise, the writer plans to publish the "Inlet Method," a "Quasi-Microscopic Method," based upon nearly 10 years of rainfall-runoff gagings in Baltimore.

The authors should be commended for stimulating interest and suggesting desirable research in urban drainage design. Undoubtedly, ideas presented in their paper will beneficially influence future research and thinking on hydrology of urban areas.



TIME DISTRIBUTION OF GREATEST 120 MINUTE DURATION EXCESSIVE PRECIPITATION RATES (ABOVE 2 YEAR FREQUENCY) AT BALTIMORE, MARYLAND (1894 – 1935 INCLUSIVE)

FIGURE 3

ACKNOWLEDGMENT

The compilation of the data of Tables I, II, and III, and other rainfall studies, runoff gagings and analyses were performed as part of the Storm Drainage Research Project at The Johns Hopkins University, Institute for Cooperative Research.

REFERENCES

1. Report on Drainage Verification at Military Establishments, Freeman Field and St. Anne Auxiliary Field, Indiana, Godman Field, Kentucky, Lockbourne Army Air Base and Wright Field, Ohio, War Dept., Corps of Engineers Ohio River Division, Louisville, Kentucky, p. 311, July 1947.

2. Seaver, Clifford, Principles of Storm Water Runoff Derived from A Study of Sewer Gagings, The Municipal Engineers Journal, Third Quarterly Issue, Vol. 35, p. 76, 1949.

3. Potter, W. H., Surface Runoff from Agricultural Watersheds, Surface Drainage, Highway Research Board, Division of Engineering and Industrial Research, National Research Council, Research Report No. 11-B, pp. 23-24, December 1950.

4. Wood, Ian R., A New Method of Urban Drainage Design, Canberra, Australia, unpublished manuscript, 1957.

5. Bock, Paul, "A Study of Urban Rainfall-Runoff Relationships," Doctoral Dissertation, School of Engineering, The Johns Hopkins University, Baltimore, Maryland, May 1958 (to be published).

6. Kuichling, Emil, "The Relation between the Rainfall and the Discharge of Sewers in Populous Districts," Transactions, ASCE, Vol. 20, p. 28, Jan-June 1889.

7. Keifer, C. J. and H. H. Chu, "Synthetic Storm Pattern for Drainage Design," Journal of the Hydraulics Division of the ASCE, Vol. 83, No. HY 4, August 1957, pp. 1332 - (9, 10, 11, 12).

8. Bock, Paul, "Progress Report of the Storm Drainage Research Project," July 1, 1955 to June 30, 1956, The Johns Hopkins University Sanitary Engineering Department, Baltimore, Maryland.

9. Bock, Paul, "Wetted Perimeter and Hydraulic Radius as Parameters in Flow Equations," presented at the Fortieth Annual Meeting of the American Geophysical Union, 1959 (to be published).

10. Izzard, C. F., "Hydraulics of Runoff from Developed Surfaces," Proceedings, Highway Research Board, National Research Council, Dec. 1946, pp. 149-150.

CARL F. IZZARD,[1] F. ASCE and CHARLES L. ARMENTROUT.[2]—The authors present a comprehensive study of a multitude of factors which may affect the rainfall-runoff relationship. While this work is a definite contribution to the field of urban drainage, it must be examined for verification and simplification of the procedure. An effort has been made by the writers to simplify the gutter-hydrograph computations and to compare the results with actual runoff hydrographs measured at inlets in residential areas.

While the Chicago Hydrograph Method provides a means of reproducing the runoff hydrograph from a given rainfall pattern, complications would arise in its application to a suburban development having drainage areas with varying shapes and slopes. Separate overland flow routing curves such as Fig. 8 would be required for variations of length or slope encountered in the design. The same would be true of gutter-storage curves such as Fig. 13.

By use of a few simplifying assumptions, the storage curves shown in Figs. 8 and 13 can be combined into one set of curves which can be used for any drainage area or gutter, regardless of grade, cross-slope, or roughness.

---

1. Chf., Div. of Hydr. Research, Bureau of Public Roads.
2. Hydr. Engr., (Research), Bureau of Public Roads.

The first step involves the detention constant, K, which is defined by Eq. (10). The use of a constant value of rainfall intensity for the determination of K simplifies the computations and introduces little discrepancy in the results. Thus, for a given storm, K would be computed for an average intensity and this value of K would be used to compute the equilibrium detention for a strip of unit width by Eq. (11).

The storage in the gutter at equilibrium is expressed by Eq. (23). Substituting the value for the cross-sectional area of flow in the gutter from Eq. (20), the gutter storage equation becomes

$$S_e = \frac{4}{7} A_e L_g \tag{23a}$$

where A_e denotes area at equilibrium. The gutter storage Eq. (25) can be expressed identically as

$$S = \frac{S_e}{Q_e^{3/4}} \left[\frac{4}{11} I_o^{3/4} + \frac{7}{11} Q_o^{3/4} \right] \tag{25a}$$

From analytical studies using the step method of routing steadily varying flow through a gutter, it was found that combining the overland flow detention and gutter storage in the storage equation would give a close approximation of the outflow hydrograph. The storage equation would become

$$S = \frac{S_e + D_e L_g}{Q_e^{3/4}} \left[\frac{4}{11} I^{3/4} + \frac{7}{11} Q^{3/4} \right] \tag{25b}$$

Using this storage value in Eq. (27) a graph similar to Fig. 13 would serve for routing both overland detention and gutter storage. In this form, however, the graph would serve for only one particular drainage area and road section. A change of slope, roughness, or length would require preparation of a new graph.

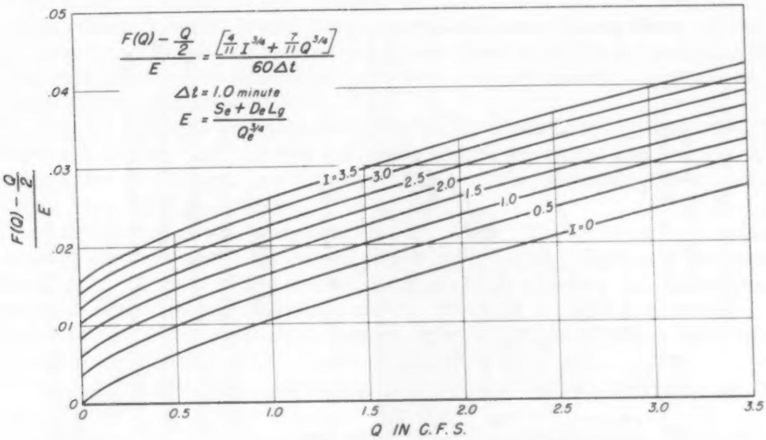
An expansion of Eq. (27) leads to a simplification

$$F(Q) = \frac{1}{2} Q + \frac{S_e + D_e L_g}{Q_e^{3/4}} \left[\frac{\frac{4}{11} I^{3/4} + \frac{7}{11} Q^{3/4}}{60 \Delta t} \right] \tag{27a}$$

The independent variables are all contained in S_e, D_e, and $Q_e^{3/4}$. By transposing,

$$\frac{F(Q) - \frac{Q}{2}}{\dfrac{S_e + D_e L_g}{Q_e^{3/4}}} = \frac{\left(\frac{4}{11} I^{3/4} + \frac{7}{11} Q^{3/4} \right)}{60 \Delta t} \tag{27b}$$

The result is a mathematical relationship of inflow and outflow which pertains to all sections. Standard storage curves can be prepared (Fig. 29) with the left side of (27b) as ordinate versus the outflow as abscissa for different values of inflow. In order to shorten terminology, $(S_e + D_e L_g)$ will be referred to as S_{DE} and $\frac{S_{DE}}{Q_e^{3/4}}$ will be referred to as E. Technically, it would be necessary



STANDARD STORAGE CURVES

FIGURE 29

to compute a value of E for each rainfall intensity encountered. However, this value does not vary greatly with differing rainfall intensities. The procedure is greatly simplified by using the same constant value of i as used in computing K and D_e. This resulting constant value of E will cause only a small percentage of error in the shape and peak of the hydrograph.

The routing Eq. (28) would not be changed by these adjustments but two columns would be added to the computations shown in Table III. After F(Q) is computed by (28), the next column would be for F(Q) - Q/2 and a second column for $\frac{F(Q) - Q/2}{E}$ which is the ordinate on the storage routing curves.

The methods presented in the authors' paper and in this discussion can be evaluated by application. Actual rainfall patterns have been routed through overland detention and gutter storage, using the Standard Storage Curves. The computed hydrograph has been compared with measured runoff at one inlet.

The field data used were obtained from the Storm Drainage Research Project conducted by The Johns Hopkins University, Baltimore, Maryland, and sponsored by Baltimore City, Baltimore County, and the Bureau of Public Roads. Runoff is measured with weir installations in the inlets. Rainfall is measured by a tipping bucket rain gage. Runoff at each inlet and rainfall at the centrally located gage are recorded on a single chart by means of a multipen recorder. The particular inlet area used was a group housing residential development in the Midwood area of Baltimore. The steep roofs drain to down-spouts which discharge onto lawns and thence to the street gutter. The drainage area, which is fairly well defined, is 400 feet long and 70 feet wide. The typical cross-section is shown in Fig. 30(C).

The overland flow detention in the gaged area was computed for four surfaces: roof, grass, walks, and street. The flow from the roof discharged onto the lawn. Sample computations indicated that routing the roof discharge uniformly across the lawn by overland flow caused the computed hydrograph to delay behind the measured runoff. Actually, it would be expected that flow from the downspouts would be somewhat concentrated and possibly flow down the walks leading to the houses. There was some erosion of the grass area

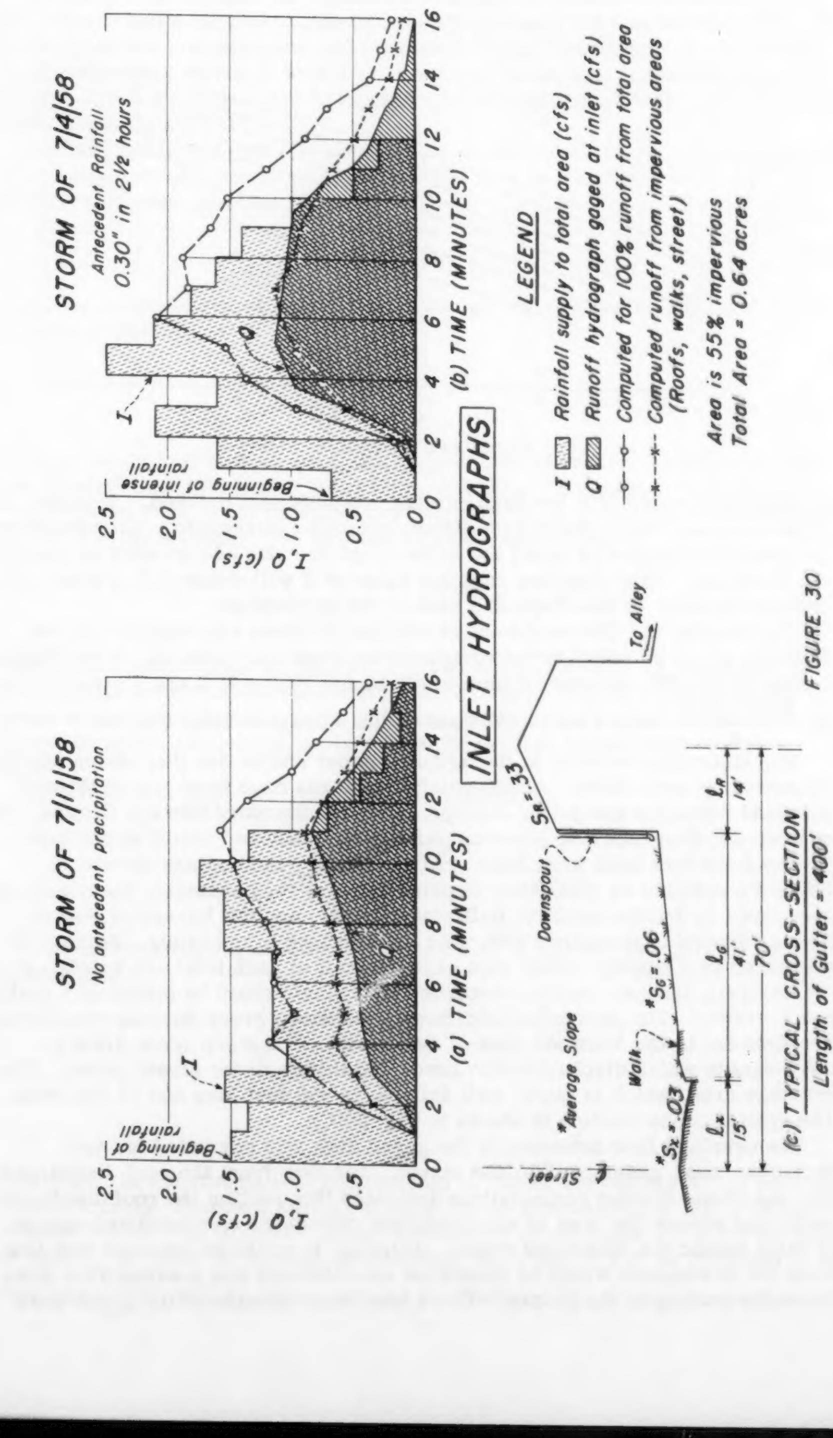

STORM OF 7/11/58
No antecedent precipitation
STORM OF 7/4/58
Antecedent Rainfall
0.30" in 2½ hours
Beginning of intense rainfall
I, Q (cfs)
(a) TIME (MINUTES)
(b) TIME (MINUTES)
INLET HYDROGRAPHS
LEGEND
I Rainfall supply to total area (cfs)
Q Runoff hydrograph gaged at inlet (cfs)
Computed for 100% runoff from total area
Computed runoff from impervious areas (Roofs, walks, street)
Area is 55% impervious
Total Area = 0.64 acres
Street
*Average Slope
Walk
Downspout
To Alley
*S_x = .03
*S_G = .06
S_R = .33
15'
41'
14'
70'
(c) TYPICAL CROSS-SECTION
Length of Gutter = 400'

FIGURE 30

adjacent to the sidewalk caused by flow which had been concentrated on the sidewalk. Consequently, it was assumed that the runoff from the roofs was added directly to the walks and the combined flow routed to the gutter. This was the only assumption used in an otherwise straightforward application of the combined routing procedure.

Eq. (11) was used to compute detention on each surface. Values of "C" used were 0.007 for roofs, 0.045 for grass, 0.012 for walks and street. A constant value of 3 inches per hour was used for i in computing the storms shown on Figs. 30(A) and 30(B).

The gutter storage was computed by Eq. (23a). The value of A_e pertains to the area of the flow at the lower end of the gutter at equilibrium. The same constant value of $i = 3$ inches per hour was used to compute Q_e, equilibrium discharge. Eqs. (19) and (20) were used to compute A_e with a value of $n = 0.012$ for a broomed concrete pavement. This value of n was derived from experimental data of the Corps of Engineers.(1)

With S_e, D_e, and Q_e computed, these values can be substituted in the routing Eq. (27b).

Hydrographs for two storms of short durations are shown in Figs. 30(A) and 30(B), one with no antecedent precipitation and the other with a small amount. Rainfall supply is shown in cubic feet per second. The computed hydrograph for 100 per cent runoff (no abstractions) is high in comparison with the actual hydrograph for Q, as might be expected. A second hydrograph was computed for each storm assuming no runoff from the grass areas. This was legitimate since infiltration and surface depression losses as presented by the authors would virtually eliminate runoff from the grass areas. With this arbitrary adjustment, the computed hydrographs for runoff agree fairly well with the observed runoff, particularly at the peak.

Other hydrographs were computed similarly which also agreed reasonably well with the observed runoff. Obviously the effect of abstractions from rainfall is important and is dependent in large measure on antecedent rainfall. In one storm lasting about two hours there was virtually 100 per cent runoff during heavy bursts. In such cases the computed hydrograph based on no abstractions reproduced that portion of the observed hydrograph.

While these tests of the method are made for only one small watershed, they do indicate that the simplified method does reproduce observed hydrographs fairly well provided the proper assumption is made regarding abstractions. In other words, it is possible by this method to route a known rainfall, or rainfall minus abstractions, through a simple drainage system to a storm drain inlet with confidence that the entire hydrograph will be substantially correct. The more precise procedure proposed by the authors would probably give a little more accuracy, but in view of the uncertainties regarding probable rainfall and abstractions, the simplified method is good enough.

The authors assume that each inlet will intercept all the water coming to it which is all right for the drainage plan in Fig. 2 when all inlets are in low points along the gutter. More commonly, some inlets will be on continuous grades where some bypass flow probably occurs at the higher rates of flow in the gutter. The writers have taken this factor into consideration in studies of flood routing along an express highway. The bypass flow was assumed to move down the gutter at a wave velocity one-third greater than the mean velocity in the reach. Flow in the storm drain was routed by the offset method used by the authors. The whole procedure can be programmed easily for electronic computation, thus making it feasible to route any given rainfall pattern through a system of storm drains along a highway.

At the present there is some doubt that the effort would be worthwhile because of recognized deficiencies in knowledge of the hydraulics of the inlets. Also, more needs to be known about the translation of flood waves in the main storm sewer, especially at junctions on steep slopes.

While the authors went to considerable pains to derive the equations for a design storm pattern having duration and intensities suitable to the Chicago area, the simplified method herein presented can be used for any desired storm pattern and abstractions.

The earlier methods of overland flow routing by Izzard, (reference 6, page 135) which were somewhat laborious, have been improved by the authors' introduction of the storage equation. While some approximations are involved, these are insignificant as demonstrated by the fact that earlier laboratory measurements of the overland flow hydrographs have been reproduced quite faithfully using a routing chart similar to Fig. 8. The writers then combined overland flow with gutter routing to further simplify the process. The results demonstrated in Fig. 30 indicate that down to the inlet, at least, the simplified method gives reasonable results.

The writers do not suggest that a design engineer should use the storage routing procedures presented herein for storm drain design. Abstractions from rainfall cannot be accounted for by such a simple procedure as eliminating runoff from the pervious area. This simplified method is a handy research tool which can be used to provide a better understanding of the relationship between rainfall and runoff. It can be used in conjunction with gaged rainfall and runoff records to study empirically the abstractions from rainfall. The computations can be performed easily and quickly with a slide rule and are also readily adaptable to programming for electronic computation.

An understanding of the rainfall runoff relationship is important to the drainage engineer. Nevertheless the solution to urban runoff on a frequency basis may lie in a statistical analysis of runoff records over a sufficient period of years rather than computing runoff from a storm having a given frequency. Unfortunately, little data are available other than The Johns Hopkins project. This situation could be remedied if progressive engineering organizations in various sections of the country should initiate accurate gaging programs in urban areas and gather sufficient date for analysis.

(Note: Reference 6 should read "Hydraulics of Runoff from Developed Surfaces," dated 1946, not 1956.)

REFERENCES

1. Airfield Drainage Structure Investigation, Corps of Engineers, St. Paul District Sub-office, Hydraulic Laboratory, Iowa City, Iowa, Hydraulic Laboratory Bulletin No. 54, April 1949.

RESISTANCE PROPERTIES OF SEDIMENT-LADEN STREAMS[a]

Discussion by Tsung-Lien Chou

TSUNG-LIEN CHOU,[1] F. ASCE.—The authors are to be highly complimented for taking an important but long neglected problem into the laboratory and the library with special technical dexterity and mathematical insight of eliminating the interwoven elements and of bringing the required factors in focus. The results are neatly tabulated and the conclusions drawn are concise and precise. Since the problem is not only important for practical hydraulic engineers in dealing with river and canal works, but also is closely related with the more theoretical and broader side of boundary layer and turbulence in general, the writer would like to bring out some points which may help clarify the issue.

On Table 4, it is shown that in each set of observation, all important hydraulic characteristics are identical except the slope s and the sediment discharge concentration $\bar{c}$. With these two, the authors attribute a direct cause - effect correlation of friction factor f_b and $\bar{c}$. As repeated on Table 5, the figures are consistent and impressive, though not quite proportional. What this amounts to is that the structure of this part of the problem depends on the accuracy and reliability of the measurement of s which is always delicate and complicated. At this junction, it may be mentioned that in empirical formulas like Darcy-Weisbach $H_f = f\frac{L}{4r}\frac{U}{2g}$ or Chezy $U = C\sqrt{rs}$, the coefficients f or c is used to fit in the observed values and they are supposed to take care of all losses which will, in modern hydrodynamical terminology, include losses clue to skin friction, form resistance and turbulence in the pure fluid side and losses in knocking out solid particles from embedment, bringing them into suspension, and accelerating them to the same velocity as the dispersion medium, and host of other losses which are yet unknown. As widely observed, the velocity u varies with space and time, even in a uniform conduit under steady flow. With the empirical relation of u and s as indicated above, synchranically together with u varies the slope s, which may include many ups and downs, even negative values in backflow pockets. In natural streams, only mean values are taken for both. Therefore, in real sense, both u and s are statistical in nature and nothing more than mathematical fictions. Similar are the friction factors, Darcy-Weisbach for Manning n. Furthermore, these factors are not universal constants as shown in the following tabulation. Thus, each friction factor has its own structure and the relations among them are not homogeneous. The fact that they can be used to calculate flow in their special forms does not necessarily mean they can represent certain factors of

a. Proc. Paper 2020, May, 1959, by Vito A. Vanoni and George N. Nomicos.
1. Hydr. Engr., Clinton Bogert Engineers, New York, N. Y.

Empirical Formulas	Chezy	Darcy-Weisbach	Hazen-Williams	Manning
Original Forms	$v = c\sqrt{rs}$	$H_f = f\frac{L}{4r}\frac{v^2}{2g}$	$v = 1.318 c_1 r^{0.63} s^{0.54}$	$v = \frac{1.486}{n} r^{2/3} s^{1/2}$
Friction Factors	c	f	c_1	n
Energy Slope	$s = \frac{v^2}{c^2 r}$	$s = \frac{v^2}{r}\left(\frac{f}{8g}\right)$	$s = \frac{v^{1.85}}{1.667 c_1^{1.85} r^{1.167}}$	$s = \frac{v^2 n^2}{2.208 r^{4/3}}$
For equal s, Ratios of Factors	$c = \sqrt{\frac{8g}{f}}$ $= 1.11 c_1^{0.925}\left(\frac{r^{0.083}}{v^{0.075}}\right)$ $= \frac{1.486}{n} r^{1/6}$	$f = \frac{8g}{c}$ $= \frac{154.5}{c_1^{1.85} r^{0.165} v^{0.15}}$ $= \frac{3.63 n^2}{r^{1/3}}$	$c_1 = \frac{c^{1.08} v^{0.081}}{1.117 r^{0.093}}$ $= \frac{15.25}{f^{0.54} r^{0.089} v^{0.081}}$ $= \frac{1.372 r^{0.091} v^{0.081}}{n^{1.08}}$	$n = \frac{1.486}{c} r^{1/6}$ $= 1.486 r^{1/6}\left(\frac{f}{8g}\right)^{1/2}$ $= \frac{1.151 r^{0.084}}{c^{0.925} v^{0.075}}$

modern hydrodynamics. Now on Table 4, f is held to count for the variation of minute vortices and others. This is apparently outside the function assigned to it in the original design.

In applying the empirical formula in a flume like this, s should cover only that portion with established normal flow, and with the entrance and tailwater regions deducted and the cross-section of observation would be set at the geometrical center of the portion. Any average value from tilting of flume bed or the differences of water levels at the ends of the flume may bring in drop-down or back water effect, which would not represent the true value of s.

By solidifying the flume bed after running with sediment, the authors tried to eliminate all factors due to bed configuration in order to emphasize the influence of sediment on hydraulic resistance. It may be worthwhile to recall that the turbulence produced on a movable bed is not the same as on similar bed which is fixed, even though the configurations are the same. On a movable bed, turbulence is evened out by picking up solid particles at points of strong intensity and dumping some of its load at points of weak intensity, or the distribution of turbulence contours is more or less conformal to the bed topography. Contrarily, on fixed bed, irregular configuration tends to amplify the turbulence at points of strong intensity and therefore greater turbulence is expected on fixed bed. Thus some part of the calculated value might be offset by this cause.

The velocity profile shown on Fig. 2 are apparently not taken from the experiments of this flume (10-1/2 inch wide). As the depth, the slope and the width are identical for the two profiles, the one with higher velocities will give higher mean velocity and thereby bigger discharge. In that case, more power is required by the flow(7) and there is hardly any strick comparison of the two.

The authors maintain that the suspended sediment reduces hydraulic roughness by its damping effect on turbulence by the following reasoning: "To keep sediment in suspension, i.e. to prevent it from settling due to gravitational force, work must come from the vertical component of turbulence

fluctuations and must result in damping of turbulent motion" and "this means that the momentum transfer coefficient is also decreased thus allowing the velocity and velocity gradient to increase."

It should be remembered that turbulence is created at the expense of energy stock of the flow. That is to say, as soon as the turbulence is generated, a certain amount of energy is lost from the flowing stream, no matter whether energy is used in keeping sediment in suspension or in supplying Vortex motion of minute masses and finally dissipated into heat. To keep sediment continuously in suspension, an uninterrupted supply of the necessary energy through turbulence is required. However this damping action may have the effect of preventing the eddies spread out into the entire stream by localizing them to the lower strata. Finally in a stream where the saturated suspension tends to dump its load, there is a chance of releasing some energy for accelerating the flow. These conditions are limited to certain part or certain period of flood where suspended load is dumped.

Furthermore, the transportation processes of solid particles are different for different effective sizes and distributions. Only fine particles less than 0.2 mm can be thrown up into suspension by and move with turbulence generated purely by hydraulic resistance. On the other hand, bigger particles create extra turbulence by their projections into flowing stream and produce suction effect helping them into sliding, rolling and saltation. This demands more energy consumption and thereby increases the hydraulic resistance. The above conditions are well observed in pumping sands in suspension and suspension tests.

In addition to the direct experimentation in laboratory, the authors collect some interesting information of field data from national streams. Here the direct cause-effect correlation of f and $\bar{c}$ is further overshadowed by additional factors of (a) source of solid supply, varying with the topographical and geological conditions of the river valley, (b) hydraulic properties of the stream, aggrading or degrading. The mutual action and reaction of these factors can easily produce contradictory effects.

In general, a rising stage has higher average velocities for two reasons. First, in rising stage, the energy slope is a drop-down curve with increasing velocities towards each downstream section. Thus the velocity is accelerated. Secondly, the potential head is converted into kinetic head with smaller loss.

In most cases, the amount and the intensity of sediment load vary with the stage. If a freshet is originated from an upstream watershed and gains strength in flowing downstream, it will pick up silt to its optimum capacity. This happens very often in the rising stage. After passing over the peak flow, the river tends to dump its suspended load. However, there may be exceptions to this rule in dealing with difference sources of solid supply. For instance, on the Yellow River at Tung-Kuang, the intensity of suspended sediment varies with the source of flood. If it comes from the north bank tributaries (Shansi Province) where steeper grades of river beds with thicker eolian deposits of loess, the River has a tendency to dump its load even in rising period. The contrary is true, if the flood is started from the south side. Similar situation is observed at Chingling Chi on the Yangtze River where the discharge from the south is clarified by passing the Tung-Ting Lake which acts as settling basin.

The above few examples just illustrate the complicated relations of various factors in natural streams and the cause-effect correlations can easily be reversed, unless comprehensive assessments are made for some period.

On the theoretical side, the sediment can reduce hydraulic resistance in the following ways:

(1) Smoothing up the surface roughness by scouring off the mound and filling up the depressing
(2) Evening off the aquatic growths
(3) Exposing fresh surfaces which are more pliable
(4) Compacting particles by selective deposition and distribution
(5) Keeping down the vertical component of turbulence (as the authors' main item). Again, every factor must be properly appraised.

The problem of suspended load hydraulic resistance is complicated but very interesting. The authors have opened up a new vista for both practical and theoretical investigation. Perhaps, another way of attack may be the direct measurements of the turbulence itself for the similar condition of turbid and clear flows.

ng

bid

PROCEEDINGS PAPERS

The technical papers published in the past year are identified by number below. Technical-division sponsorship is indicated by an abbreviation at the end of each Paper Number, the symbols referring to: Air Transport (AT), City Planning (CP), Construction (CO), Engineering Mechanics (EM), Highway (HW), Hydraulics (HY), Irrigation and Drainage (IR), Pipeline (PL), Power (PO), Sanitary Engineering (SA), Soil Mechanics and Foundations (SM), Structural (ST), Surveying and Mapping (SU), and Waterways and Harbors (WW), divisions. Papers sponsored by the Department of Conditions of Practice are identified by the symbols (PP). For titles and order coupons, refer to the appropriate issue of "Civil Engineering." Beginning with Volume 82 (January 1956) papers were published in Journals of the various Technical Divisions. To locate papers in the Journals, the symbols after the paper number are followed by a numeral designating the issue of a particular Journal in which the paper appeared. For example, Paper 1859 is identified as 1859 (HY7) which indicates that the paper is contained in the seventh issue of the Journal of the Hydraulics Division during 1958.

VOLUME 84 (1958)

SEPTEMBER: 1750(IR3), 1751(IR3), 1752(IR3), 1753(IR3), 1754(IR3), 1755(ST5), 1756(ST5), 1757(ST5), 1758 (ST5), 1759(ST5), 1760(ST5), 1761(ST5), 1762(ST5), 1763(ST5), 1764(ST5), 1765(WW4), 1766(WW4), 1767 (WW4), 1768(WW4), 1769(WW4), 1770(WW4), 1771(WW4), 1772(WW4), 1773(WW4), 1774(IR3), 1775(IR3), 1776(SA5), 1777(SA5), 1778(SA5), 1779(SA5), 1780(SA5), 1781(WW4), 1782(SA5), 1783(SA5), 1784(IR3)[c], 1785(WW4)[c], 1786(SA5)[c], 1787(ST5)[c], 1788(IR3), 1789(WW4).

OCTOBER: 1790(EM4), 1791(EM4), 1792(EM4), 1793(EM4), 1794(EM4), 1795(HW3), 1796(HW3), 1797(HW3), 1798(HW3), 1799(HW3), 1800(HW3), 1801(HW3), 1802(HW3), 1803(HW3), 1804(HW3), 1805(HW3), 1806(HY5), 1807(HY5), 1808(HY5), 1809(HY5), 1810(HY5), 1811(HY5), 1812(SM4), 1813(SM4), 1814(ST6), 1815(ST6), 1816(ST6), 1817(ST6), 1818(ST6), 1819(ST6), 1820(ST6), 1821(ST6), 1822(EM4), 1823(PO5), 1824(SM4), 1825(SM4), 1826(SM4), 1827(ST6)[c], 1828(SM4)[c], 1829(HW3)[c], 1830(PO5)[c], 1831(EM4)[c], 1832(HY5)[c].

NOVEMBER: 1833(HY6), 1834(HY6), 1835(SA6), 1836(ST7), 1837(ST7), 1838(ST7), 1839(ST7), 1840(ST7), 1841(ST7), 1842(SU3), 1843(SU3), 1844(SU3), 1845(SU3), 1846(SU3), 1847(SA6), 1848(SA6), 1849(SA6), 1850 (SA6), 1851(SA6), 1852(SA6), 1853(SA6), 1854(ST7), 1855(SA6)[c], 1856(HY6)[c], 1857(ST7)[c], 1858(SU3)[c].

DECEMBER: 1859(HY7), 1860(IR4), 1861(IR4), 1862(IR4), 1863(SM5), 1864(SM5), 1865(ST8), 1866(ST8), 1867 (ST8), 1868(PP1), 1869(PP1), 1870(PP1), 1871(PP1), 1872(PP1), 1873(WW5), 1874(WW5), 1875(WW5), 1876 (WW5), 1877(CP2), 1878(ST8), 1879(ST8), 1880(HY7)[c], 1881(SM5)[c], 1882(ST8)[c], 1883(PP1)[c], 1884(WW5)[c], 1885(CP2)[c], 1886(PO6), 1887(PO6), 1888(PO6), 1889(PO6), 1890(HY7), 1891(PP1).

VOLUME 85 (1959)

JANUARY: 1892(AT1), 1893(AT1), 1894(EM1), 1895(EM1), 1896(EM1), 1897(EM1), 1898(EM1), 1899(HW1), 1900(HW1), 1901(HY1), 1902(HY1), 1903(HY1), 1904(HY1), 1905(PL1), 1906(PL1), 1907(PL1), 1908(PL1), 1909(ST1), 1910(ST1), 1911(ST1), 1912(ST1), 1913(ST1), 1914(ST1), 1915(ST1), 1916(AT1)[c], 1917(EM1)[c], 1918(HW1)[c], 1919(HY1)[c], 1920(PL1)[c], 1921(SA1)[c], 1922(ST1)[c], 1923(EM1), 1924(HW1), 1925(HW1), 1926 (PL1), 1927(HW), 1928(HW1), 1929(SA1), 1930(SA1), 1931(SA1), 1932(SA1).

FEBRUARY: 1933(HY2), 1934(HY2), 1935(HY2), 1936(SM1), 1937(SM1), 1938(ST2), 1939(ST2), 1940(ST2), 1941(ST2), 1942(ST2), 1943(ST2), 1944(ST2), 1945(HY2), 1946(PO1), 1947(PO1), 1948(PO1), 1949(PO1), 1950(HY2)[c], 1951(SM1)[c], 1952(ST2)[c], 1953(PO1)[c], 1954(CO1), 1955(CO1), 1956(CO1), 1957(CO1), 1958(CO1), 1959(CO1).

MARCH: 1960(HY3), 1961(HY3), 1962(HY3), 1963(IR1), 1964(IR1), 1965(IR1), 1966(IR1), 1967(SA2), 1968(SA2), 1969(ST3), 1970(ST3), 1971(ST3), 1972(ST3), 1973(ST3), 1974(ST3), 1975(ST3), 1976(WW1), 1977(WW1), 1978(WW1), 1979(WW1), 1980(WW1), 1981(WW1), 1982(WW1), 1983(WW1), 1984(SA2), 1985(SA2)[c], 1986 (IR1)[c], 1987(WW1)[c], 1988(ST3)[c], 1989(HY3)[c].

APRIL: 1990(EM2), 1991(EM2), 1992(EM2), 1993(HW2), 1994(HY4), 1995(HY4), 1996(HY4), 1997(HY4), 1998 (SM2), 1999(SM2), 2000(SM2), 2001(SM2), 2002(ST4), 2003(ST4), 2004(ST4), 2005(ST4), 2006(PO2), 2007 (HW2)[c], 2008(EM2)[c], 2009(ST4)[c], 2010(SM2)[c], 2011(SM2)[c], 2012(HY4)[c], 2013(PO2)[c].

MAY: 2014(AT2), 2015(AT2), 2016(AT2), 2017(HY5), 2018(HY5), 2019(HY5), 2020(HY5), 2021(HY5), 2022(HY5), 2023(PL2), 2024(PL2), 2025(PL2), 2026(PP1), 2027(PP1), 2028(PP1), 2029(PP1), 2030(SA3), 2031(SA3), 2032(SA3), 2033(SA3), 2034(ST5), 2035(ST5), 2036(ST5), 2037(ST5), 2038(PL2), 2039(PL2), 2040(AT2)[c]. 2041(PL2)[c], 2042(PP1)[c], 2043(ST5)[c], 2044(SA3)[c], 2045(HY5)[c], 2046(PP1), 2047(PP1).

JUNE: 2048(CP1), 2049(CP1), 2050(CP1), 2051(CP1), 2052(CP1), 2053(CP1), 2054(CP1), 2055(CP1), 2056 (HY6), 2057(HY6), 2058(HY6), 2059(IR2), 2060(IR2), 2061(PO3), 2062(SM3), 2063(SM3), 2064(SM3), 2065 (ST6), 2066(WW2), 2067(WW2), 2068(WW2), 2069(WW2), 2070(WW2), 2071(WW2), 2072(CP1)[c], 2073(IR2)[c], 2074(PO3)[c], 2075(ST6)[c], 2076(HY6)[c], 2077(SM3)[c], 2078(WW2)[c].

JULY: 2079(HY7), 2080(HY7), 2081(HY7), 2082(HY7), 2083(HY7), 2084(HY7), 2085(HY7), 2086(SA4), 2087 (SA4), 2088(SA4), 2089(SA4), 2090(SA4), 2091(EM3), 2092(EM3), 2093(EM3), 2094(EM3), 2095(EM3), 2096 (EM3), 2097(HY7)[c], 2098(SA4)[c], 2099(EM3)[c], 2100(AT3), 2101(AT3), 2102(AT3), 2103(AT3), 2104(AT3), 2105(AT3), 2106(AT3), 2107(AT3), 2108(AT3), 2109(AT3), 2110(AT3), 2111(AT3), 2112(AT3), 2113(AT3), 2114(AT3), 2115(AT3), 2116(AT3), 2117(AT3), 2118(AT3), 2119(AT3), 2120(AT3), 2121(AT3), 2122(AT3), 2123(AT3), 2124(AT3), 2125(AT3).

AUGUST: 2126(HY8), 2127(HY8), 2128(HY8), 2129(HY8), 2130(PO4), 2131(PO4), 2132(PO4), 2133(PO4), 2134 (SM4), 2135(SM4), 2136(SM4), 2137(SM4), 2138(HY8)[c], 2139(PO4)[c] 2140(SM4)[c].

SEPTEMBER: 2141(CO2), 2142(CO2), 2143(CO2), 2144(HW3), 2145(HW3), 2146(HW3), 2147(HY9), 2148(HY9), 2149(HY9), 2150(HY9), 2151(IR3), 2152(ST7)[c], 2153(IR3), 2154(IR3), 2155(IR3), 2156(IR3), 2157(IR3), 2158 (IR3), 2159(IR3), 2160(IR3), 2161(SA5), 2162(SA5), 2163(ST7), 2164(ST7), 2165(SU1), 2166(SU1), 2167(WW3), 2168(WW3), 2169(WW3), 2170(WW3), 2171(WW3), 2172(WW3), 2173(WW3), 2174(WW3), 2175(WW3), 2176 (WW3), 2177(WW3), 2178(CO2)[c], 2179(IR3)[c], 2180(HW3)[c], 2181(SA5)[c], 2182(HY9)[c], 2183(SU1)[c], 2184 (WW3)[c], 2185(PP2)[c], 2186(ST7)[c], 2187(PP2), 2188(PP2).

c. Discussion of several papers, grouped by divisions.

ON

s

SA 5

SEPTEMBER 1959 — 37 NO. SA 5
VOLUME 85 PART 2

Your attention is invited

NEWS OF THE SANITARY ENGINEERING DIVISION OF ASCE

AMERICAN SOCIETY OF CIVIL ENGINEERS FOUNDED 1852

JOURNAL OF THE SANITARY ENGINEERING DIVISION
PROCEEDINGS OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS

DIVISION ACTIVITIES
SANITARY ENGINEERING DIVISION
Proceedings of the American Society of Civil Engineers

NEWS

September, 1959

PLAN TO ATTEND

the

SANITARY ENGINEERING CONFERENCE

in Cincinnati

· on

January 6-8, 1960

* *

A Change in News Editors

With the completion of this issue of the Sanitary Engineering Division News, your editor turns over the reins to John R. Thoman of Atlanta, Georgia. John Thoman is the Public Health Service's Program Director for its water pollution control activities in the Southeast. He has been a member of the Society since 1942 and an active participant in sanitary engineering affairs.

We are fortunate in having Mr. Thoman as our News Editor, and he has the right to expect the support of Division members in his efforts. Whether you are in a consulting practice, on a university staff, engaged in research, or public practice of sanitary engineering, you can serve your fellow professionals by keeping the News Editor informed on new developments in your area of interest.

I have enjoyed serving as News Editor during the past two years. The support given this activity by the Executive Committee, the Assistant Editors, Headquarters ASCE, and those of you who have individually contributed news items has been sincerely appreciated.

David H. Howells

Note: No. 1959-37 is part of the copyrighted Journal of the Sanitary Engineering Division, Proceedings of the American Society of Civil Engineers, Vol. 85, SA 5, September, 1959.

Did You Know That

George E. ("Doc") Symons has reopened his consulting office on a full-time basis. He will again offer services as a consultant in the field of sanitary engineering and as a consulting editor and writer of technical articles, brochures, bulletins, market study reports, and research and development reports. He will maintain his office in Larchmont, New York.

Lawson D. Matter, Chief of the Pennsylvania State Health Department's Water Supply Section, retired on June 30 after 40 years of uninterrupted service with the Department. Born and reared in Harrisburg, Mr. Matter graduated in 1918 from Gettysburg College as a sanitary engineer. During World War I he was a lieutenant in the U. S. Army Corps of Engineers. He joined the State Health Department in 1919. A national director of the American Water Works Association, Mr. Matter assisted in the formation of the Pennsylvania sections of AWWA, S&IWA, and Water Works Operators Association.

Wildred F. Langelier, renowned water chemist and Professor Emeritus of Sanitary Engineering of the University of California, received a fifty-year membership certificate from the American Chemical Society at its April meeting in Boston. In recognition of the service which he has rendered the profession, the California Section of the Society featured Dr. Langelier on the cover of the April issue of its official magazine, The Vortex. In an article entitiled, "Half a Century," the editor called attention to the outstanding research which won for "Bill" Langelier the Goodell and Fuller Awards of the American Water Works Association, and the Rudolph Hering Medal of the Sanitary Engineering Division, ASCE.

Fritz Sulzer has recently been appointed an Assistant Professor of Sanitary Science in the Department of Sanitary Engineering of the University of North Carolina School of Public Health. Since 1955, Dr. Sulzer has served as a Research Fellow at the Swiss Federal Institute for Water Supply and Sewage Purification, working on the microbiology of activated sludge, water disinfection and related subjects. At the University of North Carolina Dr. Sulzer will be involved in the guidance of doctoral research students and teaching in sanitary chemistry and microbiology in the Department of Sanitary Engineering.

SANITARY ENGINEERING EDUCATION

Sanitary Engineering and Environmental Control

Northwestern University has recently published a new bulletin concerning their graduate program in sanitary engineering and environmental control which evidences a real advance from the traditional concept of sanitary engineering. The idea of environmental control is a challenging one, and it is refreshing to know that a leading university is tooling up its graduate program with this imaginative horizon in sight.

The Congress has already recognized the hazards of our changing environment. A report (No. 309) from Mr. Fogarty, Chairman, House Committee on Appropriations, of April 28, 1959, stated:

> "Over the past several years a number of environmental factors affecting health have become increasingly significant. The development of industrial processes and industrial products has taken place

at a rate so rapid that direct and indirect effects on the health of the worker, the user of the processed product, and the general environment have not been adequately evaluated. The considerably expanded use and diversity of sources of radioactive products is a technical development of particular significance to health. The continued growth of gigantic metropolitan complexes has created special problems related to communicable diseases, problems of mental health, and, to a certain extent, has made it increasingly difficult to deliver health services. Related to the growth of metropolitan areas and the expansion of industrial production are the increasing problems of air and water pollution and their effects on the health of the population, which are at present inadequately understood."

Clearly, sanitary engineering education must reach beyond its present boundaries if the engineer is to assume a leadership role in solving the broad environmental problems of today and tomorrow.

The bulletin from Northwestern University states that because their graduate program in sanitary engineering is international in scope, the curriculum is designed to provide a basic education which will be useful in any part of the world. This course work is integrated with research on the frontiers of knowledge in fields such as water use, systems analysis, the disposal of radioactive and other wastes, sanitary biology and chemistry, atmospheric pollution and certain aspects of planning in order to extend the concepts of sanitary engineering to broader fields than simply the control of disease. Through this program, Northwestern graduates are prepared to practice not only as consultants and with health departments, but also with planning commissions and with organizations interested in the conservation and utilization of natural resources.

For copies of the graduate school bulletin and additional information regarding this program write: The Chairman, Civil Engineering Department, The Technological Institute, Northwestern University, Evanston, Illinois.

•

John Crerar Library Movies

The John Crerar Library, one of the world's largest collections of technical, scientific and medical literature, will move to the Technology Center campus of Illinois Institute of Technology. The Crerar Library will have custody of the Illinois Institute's technical library collection and will be located in a new public library building to be constructed on the Illinois Tech campus. The action has been approved by the boards of both organizations.

The move is being made to provide a location for the Crerar Library in a center of technological education and scientific research to make possible the expansion and improvement of its services, and to realize economies in the operation of the Library. Students, faculty members and scientific personnel of Technology Center comprise the largest single group of users of the Crerar Library at this time.

The Library will continue to be under the jurisdiction of the Crerar Board. Illinois Tech will turn over its estimated 125,000 engineering and scientific books and periodicals to the custody of the Crerar Board and to the administration of the Crerar Library staff. The Crerar Library has approximately 1,000,000 books and periodicals of all classifications. The Illinois Tech Library staff will be consolidated with the Crerar staff, resulting in a total group of approximately 85 people.

SANITARY ENGINEERING RESEARCH

International Research in Drinking Water Quality

The Division of Environmental Sanitation of the World Health Organization has just compiled a new list of investigators engaged in research on drinking water quality throughout the world. This contains the names of 136 investigators, who are working with 165 subjects of study. It represents work in 50 different laboratories or institutions in 34 countries.

The assembling of this information was recommended by the World Health Organization Study Group on International Standards for Drinking Water, which met in Geneva in 1956. The standards themselves, as proposed by this study group, have been published in 1958 by WHO in "International Standards for Drinking Water." The study group recommended 28 subjects for research. The number of investigators involved in each subject is as follows:

	Subject	Number of Investigators
1.	Effect of temperature and time of storage on bacterial densities of water	9
2.	Effect of sodium thiosulfate or other reducing agents on the bacterial population of stored samples of water	2
3.	The microfilter technique and its relation to other techniques for estimating densities of coliform bacteria	17
4.	Technique and significance of total bacterial count or standard plate count	7
5.	Study of improved media and procedures for isolation and detection of coliform bacteria in water	17
6.	Evaluation of the special inhibitory culture media proposed for the isolation of the coliform group of bacteria	7
7.	Investigation of the Eijkman procedure for isolation of E.coli in comparison with other techniques	6
8.	Investigation of techniques for the differentiation of E.coli of animal and of human origin	7
9.	Significance of the coliform group of bacteria and its subsections as indicators of pollution under varying climatic and hydrographic conditions	14
10.	Comparative investigation of the techniques for quantitative detection of bacteria of the coliform group recommended by the Ministry of Health for England and Wales and by the American Public Health Association	3
11.	Investigation of the possible growth of bacteria of the coliform group, including the faecal coli section, in natural waters and soils	11
12.	Potential significance of other bacterial indicators of pollution, including methods for their quantitative detection in water	16
13.	Significance of viruses found in sewage in relation to pollution of water, including methods for their quantitative detection	9
14.	Evaluation of the populations of pathogenic organisms, viruses, bacteriophages and coliform organisms in relation to the epidemiological data	5

Subject	Number of Investigators
15. Potential significance of bacteriophages as indicators of pollution	2
16. Investigation of the significance and value of various biological indices of pollution of water	16
17. Study of the ecological balances of plankton in water	21
18. Investigation of the quantitative significance of growths of plankton in relation to water quality	16
19. Isolation, identification and estimation of odor-and taste-producing substances in water	9
20. Analytical determination in water supplies of certain substances such as DDT, parathion, synthetic detergents and other compounds commonly used in agriculture and industry	5
21. Study of methods for the removal of nitrates from water	7
22. Development of acceptable techniques for the determination of chemical polluting substances in water	16
23. Study of sensitivity, precision and accuracy of chemical methods for the analysis of water	15
24. Development of a simple but accurate technique for the determination of fluoride in water	11
25. Further studies of the relation of fluoride in water to dental health in tropical and other areas	1
26. Development of techniques for the estimation of small concentrations of turbidity (less than 1.0 unit) in water	4
27. Determination of hydrogen-ion concentration (pH) in lightly buffered water	1
28. Development of procedures for the measurement of radioactive contamination of water, and consideration of the significance of limiting concentrations of radioactivity	9

It is recognized that many names and many items of study have probably been omitted. To extend this list, WHO would appreciate further information on drinking water quality research from all available sources. Copies of the list and correspondence related to research in the field of drinking water quality should be addressed to the Division of Environmental Sanitation, WHO, Geneva.

New Research at the University of Florida

A four-year study of the uptake and accumulation of radioactive substances by edible shellfish and crustaceans has recently been undertaken at the University of Florida with approval of a $95,000 research grant from the National Institutes of Health, Public Health Service. Special emphasis is to be placed on such wastes as result from the operation of nuclear powered vessels. For food it is intended to use plankton which itself has accumulated radioactivity in order to determine whether the experimental animals acquire radioactivity from the water in which they live, the food they eat, or both.

Another research grant of $10,000 has been allocated by the Portland Cement Association to study the factors which cause concrete sewers to fail

in some locations while they function satisfactorily in others. Some of the matters under study are temperature effects, flow rates, and H_2S formation. The condition of the sewage under existing climatic and topographical conditions will be studied in municipal sewer systems.

The filtration of settled sewage through sand of uniform size is also being studied. Several effective sizes will be used to determine the efficiency of each size. Twenty-six beds with depths varying from 8 to 36 inches are being operated at rates of 50,000 to 400,000 gpad. The results of this investigation will provide further data regarding sand filtration of sewage and will be a valuable corollary to results on other sand filter studies published in 1954-55.

A new grant of $39,054 was recently received from the National Institutes of Health for continued work in the chemistry of water coagulation. This is the third Public Health Service grant for this basic project, the total being $122,958.

WATER SUPPLY AND WATER POLLUTION CONTROL

World Health Organization Water Supply Program

On May 26, 1959, the Programme and Budget Committee of the Twelfth World Health Assembly unanimously adopted a resolution approving a spearhead programme, put forward by WHO's Director-General, Dr. M. G. Candau, to provide safe and adequate supplies of water to inhabitants of communities. This was recognized to be an important measure for the protection and improvement of health and indispensable for economic and social development.

According to Dr. Candau, "water predominates as the major constituent in practically every phase of an individual's physical, social and economic life. Experience has proved that making potable water available to the individual is the foundation on which rests the health and economic progress of the community. Because of the basic public health importance of community water supplies, WHO cannot remain aloof from its obligation to supply the stimulation and assistance needed to bring about their construction."

Dr. Candau also stated: "It is disheartening to record that in 1959 in many major cities and their densely urbanized satellites many millions of people are still dependent upon individual wells, springs or itinerant purveyors for this life-giving commodity. Cities, ranging from two to seven or eight million people, not only fail to furnish water through pipes to households of several hundreds of thousands of their inhabitants, but even to those directly connected to the system they supply an unsafe water, often on a rationed basis of a third of each day or less. This significant fact is often ignored in determining environmental sanitation programmes.

"The labour involved in drawing water and transporting it for a long distance, a task which often falls to the lot of the women, results in their virtual enslavement. Frequently as much as one-half of their time, day after day, month after month, is taken up with this essential chore. A very simple calculation will show that there is no more efficient means of transporting water than by a pipe. A small pipe, one inch in diameter, will deliver in a day, without human effort, as much water as can be carried by 150 women working steadily for eight hours. Even in the most advanced countries there are still great deficiences."

The spearhead programme proposed by WHO, can be summarized as follows: "Member States, with the leadership of WHO, might focus upon a

programme to bring to the houses of the people of the world safe drinking water in ample quantities. A concerted drive on this major programme would bring great results. Such an effort would not exclude all other sanitation activities, but it would place first things first. It would dramatize an overwhelming need and desire of all people. It would not be neecssary, in this programme, to waste precious years to educate people toward a desire for better water. Actually, people are ahead of the experts on this point. The practitioners should catch up with the people.

"For such a programme to be successful it should move rapidly beyond lip service to real effort, including budgetary allotments, increases in skilled personnel, and actual operating programmes. Its success would demand of ministries of health in a militant and continuing leadership and a far closer co-operation with departments of public works than now generally exists. It is sound to separate the stimulative functions of a health department from the executive functions of a public works department. Such administrative separation, however, does not justify each in going its own way. Their co-operation is essential in carrying forward a water supply programme."

The resolution adopted by the Programme and Budget Committee asked the Director-General "to continue his study of ways and means of assisting governments to provide safe and adequate supplies of water to inhabitants of their communities, including an investigation of existing international loan or other funds which might be available for investment in such facilities."

This resolution also asked the Member States of the World Health Organization to give priority in their national programmes to the construction and extension of supplies of safe water to communities, and suggested "that within each country requiring such a facility a revolving fund be established to provide loans for water supply development to local agencies of governments."

The Committee also decided that a special account should be created which would be financed by contributions to help governments to draw up plans for the provision of public water supplies.

Finally, the Committee invited "all-multilateral and bilateral agencies having an interest in this field to co-operate with the World Health Organization in carrying out a global water supply programme."

Natural Fluoride Content of Public Water Supplies

The Dental Directors of all of the State health agencies have cooperated with the Public Health Service in the assembly of data on the amount of fluoride naturally present in community water supplies throughout the country.

Water supplies of 1,903 cities and towns with a combined population of 7 million contain enough fluoride naturally to prevent two out of three dental cavities. Communities in 43 States use water in which the decay-preventing fluroide occurs naturally. Community populations served range from less than 50 to more than 500,000.

In Texas, 2,700,000 persons in 356 towns use naturally fluoridated water. In New Mexico, 465,000 people or 68 percent of the total population live in communities with water supplies naturally containing fluoride.

More than 450,000 people in 136 Illinois towns and 406,000 in 184 Iowa communities drink water with fluoride added by nature. At least 100,000 people in each of ten states-California, Colorado, Florida, Idaho, Indiana, Kansas, Louisiana, Michigan, Ohio, and Wisconsin—live in towns served by naturally fluoridated water supplies.

Thirty-five percent of the 7,000,000 persons using naturally fluoridated water live in towns and cities with populations of more than 50,000. Thirty-eight percent live in towns of from 5,000 to 50,000, and 27 percent in communities of under 1,000.

The 1,800 cities now using controlled fluoridation adjust the fluoride content to that found in a great many of the naturally fluoridated water supplies throughout the country, or from 0.7 to 1.2 parts of fluoride per million parts of water.

The 35 million people living in these 1,800 communities plus the 7 million using naturally fluoridated water means that one out of every three people using central water supplies now drinks water that has been fluoridated by nature or by the community.

Copies of the new report, "Natural Fluoride Content of Communal Water Supplies in the United States, "Public Health Service Publication No. 655, may be obtained from the U. S. Government Printing Office, Washington 25, D. C.

News From the Office of Saline Water

Freeport, Texas, has been selected as the site for the Gulf Coast saline water conversion demonstration plant authorized by Public Law 85-883. This plant is to be located about two miles from the Coast adjacent to an existing Dow Chemical plant. A sea water intake line serving the chemical plant will provide the conversion plant with feed water. Dow Chemical will sell the plant steam and will, in turn, purchase all the product water.

Interior Secretary Seaton also announced selection of an electrodialysis process for the third saline water conversion demonstration plant. This plant will be designed to convern brackish water to fresh at an anticipated rate of at least 250,000 gallons per day. It will be located in the Northern Great Plains or in the arid areas of the Southwest. The Office of Saline Water has been active in electrodialysis developments since 1953. Present research and development is being carried out at the Denver laboratories of the Bureau of Reclamation. The office has sponsored extensive testing and evaluation of experimental electrodialysis equipment. Cooperation with industrial, engineering, and research agencies, both foreign and domestic, active in electrodialysis is maintained. Several plants utilizing electrodialysis, with conversion capacities ranging up to 86,400 gallons per day, are in commercial operation. A 3 million gallon per day plant is under construction in South Africa. Operating data from these plants together with information obtained from pilot plant tests and fundamental research continue to develop improvements in electrodialysis processes.

Sewage Stabilization Ponds in Wisconsin

The Wisconsin Committee on Water Pollution recently published a report of its biological and chemical investigations of sewage stabilization ponds conducted from April, 1957, to August, 1958. The investigators were Kenneth M. Mackenthun and Clarence D. McNabb, public health biologists. The report is entitled, "Sewage Stabilization Ponds in Wisconsin," Bulletin No. WP 105. Correspondence regarding this report should be directed to Mr. Theodore F. Wisniewski, Director, Wisconsin Committee on Water Pollution, Madison, Wisconsin.

The report discloses that Wisconsin became interested in sewage stabilization ponds in 1955 and that four units were constructed during the latter part of 1956. These are the installations involved in this study.

During the 15-month period of the investigation, the ponds received settled sewage and trickling filter treated sewage at loadings ranging from 5 to 55 pounds of BOD per 100,000 square feet per day. The efficiency of the ponds at this latitude was found to be comparable with, and, during optimum climatic conditions somewhat superior to conventional secondary treatment. BOD removal ranged from 65 per cent in late winter to about 90 per cent during the summer months. No BOD reduction was found to occur in the pond receiving trickling filter effluent. Reductions in coliform organisms were greater than 98 per cent more than 87 per cent of the time. There was greater reduction in summer than under winter conditions, and, except for one sample, summermertime reductions were 99 per cent or greater.

Stream Aeration by Turbines

The fourth of a series of reports covering joint activities of the Wisconsin Committee on Water Pollution and the Sulphite Pulp Manufacturer's Research League has been released by the Wisconsin Committee on Water Pollution. Entitled, "Cooperative State-Industry Studies of Wisconsin Rivers in 1958 - Turbine Reaeration on the Flambeau, Wisconsin, and Fox Rivers," (Bu. WP 106) this bulletin reports the results of studies to evaluate the effect of turbine aeration on dissolved oxygen of streams.

The admittance of air to a turbine for the purpose of introducing oxygen into the water was found not to present any serious installational or operational difficulties providing a suction head exists. Findings at one project where no suction head existed indicated that turbines set at tailwater level may be employed for forced air introduction into the stream. Careful upstream sampling procedures were found to be necessary to evaluate the true gain in dissolved oxygen. Photosynthesis, surface reaeration, and benthic decomposition caused oxygen stratification.

Within limits, the dissolved oxygen increase through turbine reaeration was directly related to air inlet rates. Indications were that specific turbines may have a maximum oxygen solution capacity. Excess air may reduce the amount of oxygen absorbed because of altered draft tube mixing characteristics and enlarged bubble size. The range of oxygen solution efficiency for five projects of record varied from 15.2 per cent to 34.9 per cent.

Introducing air into the turbines resulted in a loss of power production in a range of 1.62 to 3.95 pounds of dissolved oxygen gain per kw-h lost.

Correspondence relative to this report should be directed to Mr. Theodore F. Wisniewski, Director, Wisconsin Committee on Water Pollution, Madison, Wisconsin.

New Water Pollution Control Legislation

The Governor of Colorado has signed into law the amendment to the State Water Pollution Control law authorizing standards for waste effluents. The new law broadens the responsibility of the State Department of Public Health and includes the provision for surveillance of water quality by the Department.

The Idaho State Board of Health adopted a water pollution control code designed to safeguard surface and ground water supplies from contamination.

The regulations include minimum treatment requirements for any waste to be removal of readily settleable and floatable solids, plus effective disinfection of domestic wastes.

Recently approved amendments to the Maine Water Pollution Control Law provide State aid to municipalities for sewage pollution surveys, include increased penalty provisions, and make the discharge of sewage, industrial, and other waste into unclassified waters unlawful without a license after September 1, 1959 (except for municipal and local subdivision facilities existing prior to that date).

Rodenticide Cited as Hazard to Water Supplies

The State Health Officer of Pennsylvania, Dr. C. L. Wilbar, Jr., has warned that there are reports of incidents in which amounts of the highly-toxic "Endrin," widely used to destroy orchard mice, have entered Pennsylvania streams. This has resulted in fish mortality and death of livestock. In at least one instance a public water supply was endangered. He stated that the indiscriminate use of this poison means an ever-present threat to persons using water likely to be contaminated by spills or runoff of "Endrin."

Our Growing Water Problems

The National Wildlife Federation has recently published an effective booklet entitled, "Our Growing Water Problems," by R. G. Lynch, a well-known newspaperman. This covers water problems, water use and supply, remedies, water laws, waste disposal, and the present status of water problems throughout the country. Copies can be obtained from the National Wildlife Federation, 232 Carroll Street, N.W., Washington, D. C.

New Engineering Text

Dr. Ven Te Chow, Professor of Hydraulic Engineering and Head, Hydraulic Engineering Division, University of Illinois, has announced the publication of his text "Open-Channel Hydraulics." This is the first English language book on this subject in 18 years. It covers basic principles, uniform flow, gradually varied flow, rapidly varied flow, and unsteady flow. The publisher, McGraw-Hill Book Company, states that this book will serve as a compendium for practicing engineers and as a source of information to scientific-minded hydraulicians as well as a textbook.

AIR POLLUTION

St. Louis and Dallas Health Departments Taking Over Air Pollution Control

Plans are now under way to consolidate into the St. Louis Health Department the activities carried on by the City Department of Smoke Regulation. It is anticipated that as a result a broad air pollution control program will be undertaken. The Dallas City Health Department proposes to take steps leading to the enactment of an air pollution ordinance placing responsibility for air pollution control in that department. This too foreshadows a revitalization of a program now dormant in another city department.

Air Pollution Prevention Program Considered in New Orleans

Growing interest is reported from Louisiana, on the part of the State Health Department, the City of New Orleans, and the New Orleans Chamber of Commerce, in the need for a control program to prevent an air pollution problem from developing as a result of the mushrooming industrial development in that area. A State Enabling Act to deal with this situation is being considered.

National Capitol Gets a Sample of California-Type Smog

On June 10, pronounced eye irritation due to air pollution in the Washington, D. C., area was reported to the Public Health Service, the District Health Department, and the District Smoke Abatement Engineer, from such varied sources as the Capitol Building, the National Library of Medicine, the Government Printing Office, the Library of Congress, and the Ketchum Elementary School, as well as from many individual citizens. According to Weather Bureau officials, this was due to a "subsidence inversion" (in which the upper air is warmer than the air near the surface) accompanied by low wind velocity. In this instance, the inversion persisted for about three days. Washington lies within the area of the Country (East of the Rockies) in which these conditions are most likely to prevail, that is, the section adjacent to the Appalachians from southwest Pennsylvania to northwest Georgia. The season of greatest prevalence is in late summer and early fall, with a secondary maximum in late spring and early summer. Special air samples were collected for the Public Health Service by the D. C. Health Department and the National Bureau of Standards.

New York and New Jersey to Cooperate on Air Pollution Problems

Four New York-New Jersey public agencies have joined forces to organize a Cooperative Committee on Interstate Air Pollution. Objectives of the Committee are to exchange complaints and information on sources, technical data, and research on interstate air pollution, and to conduct joint studies on the problem. Mr. William R. Bradley, Chairman of the New Jersey State Air Pollution Control Commission, was named Chairman of the Committee and Mr. William A. Munroe, Chief of the New Jersey State Health Department's Air Sanitation Program, was appointed Secretary. Other agencies represented on the Committee are the New York City Department of Air Pollution Control and the New York State Air Pollution Control Board.

Development of a Mass Loading and Radioactivity Analyzer

A prototype mass loading and radioactivity analyzer for atmospheric particulates has been developed by the Air Pollution Engineering Program of the Public Health Service. Beta gage technique utilizing carbon-14 is applied to measurement of the thickness of particulate deposits collected on membrane tape. This technique for mass loading analysis is also applicable to paper tape and other media of comparable thickness. The result is expressed in terms of concentration values, i.e., micrograms of particulate per cubic meter of air. The proportional counter in the instrument also measures the Alpha and Beta radioactivity content of the dust deposit and the concentration values are in terms of micro-microcuries of activity per cubic meter of air.

Preliminary indications are that mass loading analysis can be made of particulate concentrations within 100 micrograms per cubic meter, but additional empirical data will be required to assess the effect of combined errors before the limit of sensitivity can be established. Concentrations of natural radioactivity by Alpha detection can be measured as low as 35 micro-microcuries per cubic meter and at 18 micro-microcuries per cubic meter, both values being well within recommended maximum permissible levels in air.

Atmospheric Pollution

15,000 Women's Clubs Urge Concerted Action on Air Pollution

At its National Convention held in Los Angeles, June 1-3, the General Federation of Women's Clubs declared its "deep concern about the dangers of air pollution now threatening communities of the nation." Their formal resolution concluded by urging "that immediate action be taken by the U. S. Department of Health, Education, and Welfare, State and local governments, industries, and individuals for more active participation in the solutions to air pollution problems." The G.F.W.C. represents some 15,000 women's clubs in the United States alone.

Gradual Control of Olefins in Gasoline Begins in Los Angeles

Rule 63, which has been considered for some time by the Los Angeles County Air Pollution Control District, was adopted on June 16 after a public hearing. Rule 63 specifies the step-by-step control of olefins in gasoline fuel to "bromine No. 30" (approximately 16 3/4% olefin) in one year, and to "bromine No. 20" (approximately 12-1/2% olefin) in 2-1/2 years. The Los Angeles District maintains that this rule is a stopgap to cut back eye irritation and other smog effects until an automobile device or some other major control is available and will result in significant alterations in refining procedures in the Los Angeles area over the next 2-1/2 years.

Air Pollution Control Association Meets in Los Angeles

Attending the recent week-long meeting of APCA in Los Angeles were more than 600 air pollution experts—engineers, scientists, and government officials—from Canada and Mexico as well as the United States.

In response to one speaker's comment that the Federal Government should not interfere with air pollution activities, California State Senator Richard Richards said that "air pollution is no respecter of political boundaries and we need help from every agency and individual."

Fred L. Hartley, Vice President for Research of the Union Oil Company, reported the development of engine modifications which can reduce evaporation losses from carburator vents and gasoline tanks by 90% or more. Since it has been estimated that some 30% of the hydrocarbon emission reaching the air in the Los Angeles metropolitan basin came from these sources, the new device was hailed as an important step toward a "smogless" automobile. The only remaining vents to be controlled are the crankcase and the exhaust, and Mr. Hartley said that the former presents no serious problem and that control of the latter is on the way now.

Additional Air Quality Standards in California

Following the auto exhaust standards previously reported, the California Legislature has now required the State Department of Public Health to develop and publish, before February 1, 1960, standards for the quality of the ambient air of that State which shall "reflect the relationship between the intensity and composition of air pollution and the health, illness, including irritation to the senses, and death of human beings, as well as damage to vegetation and interference with visibility."

Idaho Air Pollution Commission Appointed

Governor Robert E. Smylie this month appointed Idaho's first Air Pollution Commission under provisions of an air pollution control law passed by the last Legislature. The law created the commission as a new administrative department under the State Board of Health, with the power to develop "a comprehensive program for the prevention and control of all sources of air pollution in the state." The commission also is charged with conducting research, and with supervising educational activities designed to help cut down air pollution. Public hearings are provided before any code of regulations can be placed in effect.

New Publications

The Proceedings of the Third Annual Symposium on Problems in Air Pollution held at the Franklin Institute, October 21, 1958, are now available at \$3.00 per copy. The theme of the symposium was "Odor-Measurement and Control." Franklin Institute is located at 20th and Benjamin Franklin Parkway, Philadelphia 3, Pa.

The Third Edition (1959) of INSTRUMENTS FOR THE STUDY OF ATMOSPHERIC POLLUTION, a publication by Subcommittee No. 2 of the Committee on Air Pollution Controls, is available for \$2.00 from the ASME Order Department, 29 West 39th Street, New York 18, New York. Hundreds of devices ranging from samplers to gas chromatographic equipment are listed, together with the names and addresses of suppliers.

Estimated costs for building and operating various types of equipment to remove harmful sulfur dioxide from flue gases of coal-burning power plants in the United States have been listed by the Bureau of Mines, Department of the Interior, as part of a study for the Public Health Service. The study centered on power plants of 120,000 kilowatts capacity burning an estimated 475,000 tons of coal a year. Costs of installing scrubbing equipment to curb sulfur dioxide would range from \$.75 million to \$5 million, depending on the gas-cleaning process used. Operating costs were estimated to range from \$1.40 to \$2.20 per ton of coal consumed. The operating cost would be lower if marketable by-products were recovered from the furnace gases. Three different liquid processes for removing sulfur dioxide were listed by the Bureau: the limestone, ammonia, and the sodium sulfite method. Without considering credit for by-products, the Bureau says that capital investment and operating costs were lowest for the limestone process. A copy of the publication, Report of Investigations 5469, "Cost Estimates of Liquid Scrubbing Processes for Removing Sulfur Dioxide from Flue Gases," can be obtained by writing the Publications Distribution Section, Bureau of Mines, 4800 Forbes Ave., Pittsburgh 13, Pa.

New Motion Picture

A new motion picture film on air pollution–its causes, effects and cures–has been released by the Kaiser Steel Corporation for general showing. The film is titled, "Air Pollution, Everyone's Problem." In full color and sound, the twenty-minute film was produced by the Cinematography Department in cooperation with the Air Control and Research Department. Almost a year in preparation, the film clearly explains or demonstrates the terms so commonly used in newspaper air pollution stories. "Inversion," "shear lines," "hydrocarbons," and many other related phenomena are covered in relation to the total problem. After discussing the theories of the composition of "smog," and illustrating many of the sources of air pollution not even considered by the average citizen, the film describes the work of the Air Control and Research Department at Kaiser Steel. Specific examples of how air pollution problems have been solved at the Fontana plant are detailed. An interesting sequence in the film shows how an electrostatic precipitator works, using animated film techniques. The film may be booked for viewing by organizations by addressing Public Relations Department, Kaiser Steel Corporation, P. O. Box 626, San Bernardino, California.

NUCLEAR SCIENCE

Disposal of Radioactive Wastes at Sea

A panel of the National Academy of Sciences - National Research Council - expressed the opinion recently that "certain Atlantic and Gulf of Mexico coastal areas can be used as receiving waters for the controlled disposal of packaged, low-level radioactive wastes."

The panel emphasized that by limiting the scope of its recommendations to "low-level wastes," it was considering only the safe disposal of wastes containing up to the equivalent of millicurie quantities of radioactivity per gallon, as distinguished from the high-level wastes obtained from the processing of reactor fuel elements, which may contain many hundreds of curies per gallon. Low-level wastes are composed, in the main, of the trash of industrial and academic research laboratories, hospitals, and research institutions that have been licensed by the U. S. Atomic Energy Commission to use small quantities of radioactive materials for scientific research or the diagnosis and treatment of illness.

The panel, under the chairmanship of Dayton E. Carritt, associate professor of oceanography, The Johns Hopkins University, was organized as a working group of the Academy-Research Council's Committee on Oceanography at the request of the Committee's three government sponsors–the Bureau of Commercial Fisheries, the Atomic Energy Commission, and the Office of Naval Research. The National Science Foundation later became a fourth sponsor.

Called together by Dr. Roger Refelle, director of the Scripps Institution of Oceanography, a member of the Committee on Oceanography, and chairman of the Academy's Committee on Effects of Atomic Radiation on Oceanography and Fisheries, the working group was asked to consider the levels of radioactive wastes that can be disposed of safely in coastal areas, the kinds of packaging that should be used, and to suggest specific coastal disposal sites.

In its report, the panel recognized the necessity for spotting such disposal areas at many points along the Atlantic and Gulf coasts in order to accommodate the increasing use of low-level radioactive materials as a greater number of locations.

After intensive study of local oceanographic conditions, the panel was able to recommend 28 possible disposal sites and the amount of various radioactive isotypes that could be safely dumped there in suitable containers. The 28 recommended sites included every major seaport area from Boston, Mass., to Corpus Christi, Texas, whose offshore currents and other conditions would permit safe disposal.

Some of the safety factors considered by the panel during its deliberations were the following:

1. Effects of local conditions;
2. Effective containment in concrete mixtures encased in steel drums;
3. Uptake by fish and bottom-dwelling organisms;
4. Recreational and other uses of local marine resources.

The panel also noted that in order to insure an adequate safety factor in regard to human health, it had assumed that coastal populations subsisted entirely on a sea-food diet as a source of protein.

The panel insisted, however, that before the beginning of any such shallow-depth disposal operation, an on-site survey of the area must be made to determine details of local circulation and marine biology, especially bottom-feeding organisms.

The panel further continued that disposal areas should be periodically monitored during use for changes in background radiation throughout the region, including studies of the bottom sediments and living organisms.

Treatment Plant Design Manual Errata

Two corrections in the joint ASCE-FSIWA Manual of Engineering Practice No. 36, "Sewage Treatment Plant Design" have been brought to the attention of readers (July 1959 Newsletter of the Sanitary Engineering Division). Those now using the manual are urged to notify either organization if other errors are found. With this information an errata sheet can be prepared and included in all manuals sold after its issuance. The same errata sheet would be available to those who made early purchases of the manual.

To be included in the errata sheet, all corrections must be received by either organization prior to November 1.

David H. Howells, EDITOR
11103 Dodson Lane
Silver Spring, Maryland

ASSISTANT EDITORS

Mr. Walter A. Lyon, Director
Division of Sanitary Engineering
Pennsylvania Dept. Health
P. O. Box 90, Harrisburg

Professor Gilbert H. Dunstan
Dept. Civil Engineering
Washington State Institute of Technology
Pullman, Washington

Mr. James H. McDermott
Ass't. San. Engineer
WS and WPC Program
Robert A. Taft San. Engr. Center
4676 Columbia Parkway
Cincinnati 26, Ohio

3